US Army Corps of Engineers

Sacramento District

Lower San Joaquin River Feasibility Study

San Joaquin County, California

HYDROLOGY OFFICE SUMMARY REPORT

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LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY
HYDROLOGY OFFICE REPORT, April 2014

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LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

SAN JOAQUIN COUNTY, CALIFORNIA
HYDROLOGY OFFICE REPORT

February 2014

1.0 PURPOSE OF STUDY

The purpose of this hydrology report is to perform a hydrologic analysis of the lower San Joaquin River and tributaries that impact flooding in the Lathrop and Stockton urban areas. Due to the variety of watersheds in the study area, a number of methods were utilized for each watershed analysis.

The Lower San Joaquin River feasibility study will develop flood risk management (FRM) and ecosystem restoration (EC) plans along the Lower San Joaquin River, and the Bear Creek, Mosher Slough, Calaveras River and Mormon Slough, Littlejohn Creek, Duck Creek, and French Camp Slough. New Hogan Dam on the Calaveras River and Farmington Dam on Littlejohn Creek are both Corps owned and operated flood control projects that provide flood protection and water supply and recreation to the Stockton area. The authority for the U.S. Army Corps of Engineers (USACE or Corps) to study FRM and related water resources problems in the San Joaquin River Basin, including the study area in San Joaquin County, is provided in the Flood Control Act of 1962 (Public Law 87-874).

2.0. HOW TO NAVIGATE REPORT

Appendix 1 is the Calaveras River watershed above Bellota. Appendix 2 is the Littlejohn Creek above Farmington, Ca. Appendix 3 covers Bear Creek, Mosher Slough, lower Calaveras River watershed below Bellota, and French Camp Slough watershed below Farmington, Ca.

3.0. STUDY AREA

The study area from the Reconnaissance Report, Section 905(b) Analysis, for the LSJRFS is along the lower (northern) portion of the San Joaquin River system in the Central Valley of California. The San Joaquin River originates on the western slope of the, Sierra Nevada and emerges from the foothills at Friant Dam. The river flows west to the Central Valley, where it is joined by the Fresno, Chowchilla, Merced, Tuolumne, Stanislaus and Calaveras rivers, and smaller tributaries as it flows north to the Sacramento-San Joaquin Delta. The primary study area as described in the Section 905(b) Analysis includes the main stem of the San Joaquin River and its floodplains from the Mariposa Bypass downstream to the city of Stockton. This includes the distributor channels of the San Joaquin River in the southernmost reaches of the Delta: Paradise Cut and Old River as far north as Tracy Boulevard and Middle River as far north as Victoria Canal.

On the basis of continued coordination with local interests along the San Joaquin River, the primary study area for the LSJRFS will also include the Littlejohns Creek and Farmington Dam areas southeast of Stockton, the city of Stockton extending from the Calaveras River, Mormon Slough, and Bear Creek, and tributaries north of Stockton including the Lodi WWTP at Thornton Road and Interstate 5. An overview of the San Joaquin River Basin showing reservoirs and primary gaging station locations is included in plate 1.

The overall study area includes those areas adjacent to the primary study area which could be influenced by potential actions to address the identified problems and needs.

The study area was decreased in size to the area shown in plate 2 in 2011. The area south of the Stanislaus River confluence with the San Joaquin River was excluded because the Corps is prohibited from promoting development in floodplains which is the criteria on wise use of floodplains. Some of the area to the west of the San Joaquin River is part of the Sacramento – San Joaquin River Delta and overlaps the Delta Islands Feasibility study.

A map of the study area is shown in plate 2. Plate 3 shows the boundary of San Joaquin county. It shows that the entire study area is within the San Joaquin County boundary. Plate 4 shows the boundary of the San Joaquin Area Flood Control Agency (SJAFCA). The study area extends to the south to the Stanislaus River, to the east to Jack Tone Road, and outside the SJAFCA boundary north to the Lodi WWTP. The study area covers approximately 306 square miles and is approximately 15 miles east-west and 25 miles north-south. The study area includes the communities of Stockton, Manteca, Lathrop, Lockeford, and the census designated places (CDP) of Lincoln Village, French Camp, and parts of Lodi, and Ripon. Table 1 showing the population from the 2010-2000 US census is shown below. A plot of the San Joaquin County and City of Stockton population from 1960 to 2010 and projected population to 2070 is shown in plate 6.

Table 1. 2000 and 2010 Population and Projections

2010 - 2000 Census Population within study area			
Community	2010 Population	2000 Population	Change from 2000
French Camp, CDP	3,376	4,109	-17.8%
Lathrop	18,023	10,445	72.6%
Lincoln Village, CDP	4,381	4,216	3.9%
Lodi	62,134	56,999	9.0%
Manteca	67,096	49,258	36.2%
Ripon	14,297	10,146	40.9%
Stockton	291,707	243,771	19.7%
Unincorporated County	224,292	184,654	21.5%
San Joaquin County	685,306	563,598	21.6%
Source: US Census Bureau. CDP = Census Designated Place			

Table 2. Interim Projections For California and Counties

Interim Projections for California and Counties: July 1, 2015 to 2050 in 5-year Increments. Source: CA Dept of Finance, Demographics										
County	**Estimates**		**Projections**							
	2000	**2010**	**2015**	**2020**	**2025**	**2030**	**2035**	**2040**	**2045**	**2050**
San Joaquin	567,753	686,651	739,224	795,631	862,496	935,709	1,015,876	1,100,119	1,190,107	1,288,854

4.0. STUDY AREA BASINS – GENERAL DESCRIPTION

A list of the flood control dams and reservoirs above the Stockton metro area is shown in the table 10 below entitled “Dams and Lakes in the San Joaquin River Basin”.

Table 12 shows the drainage areas within the San Joaquin River basin. Flood control projects and principle control points are described below with the percentage of the total drainage area controlled. This table shows that there is approximately 56-percent of the basin controlled at Vernalis.

Flow frequency of New Hogan dam (NHG), the Bellota control point (MRS), and Farmington dam (FRM) and the at Farmington control point (FRG) were estimated by detailed study methods using gage records on the Calaveras River for New Hogan dam and Bellota, and on Littlejohn Creek for Farmington dam and at Farmington. Frequency curves and hydrographs of unregulated flow were developed for the 50% (1/2) ACE to 0.2% (1/200) ACE events. Additional details of the Calaveras River above Bellota and Littlejohn Creek above Farmington control points may be found in the Calaveras River and Littlejohn Creek frequency analysis and hydrographs by David Ford Consulting Engineers (Ford) in June 2011 for the Lower San Joaquin River Feasibility Study [6 & 7].

Flow frequency for stream reaches downstream of the Bellota control point on the Calaveras River, and below the Farmington control point on Littlejohn Creek were developed by detailed methods using an HEC-HMS rainfall-runoff model calibrated to specific flood events. That includes the Mormon Slough which is tributary to the Calaveras River. And, the HEC-HMS model of the Littlejohn Creek watershed also includes, Duck Creek, Lone Tree Creek, and French Camp Slough. HEC-HMS models were also developed for Bear Creek and Mosher Slough watersheds, which are unregulated watersheds, and are tributary to the Delta. Additional details of the Calaveras River below Bellota and Littlejohn Creek below Farmington control points may be found in the F3 Hydrology Appendix for the Lower San Joaquin River Feasibility Study done by Peterson-Brustad, Inc Consulting Engineers (PBI) as work-in-kind for the San Joaquin Area Flood Control Agency (SJAFCA).

4.1. Bear Creek HEC-HMS Modeling General

Bear Creek is located near the city of Stockton in San Joaquin County, California plates 29 and 30 (Figure 3-2 and 3-12). The watershed runs east from the city of Stockton into the Sierra Nevada foothills in Calaveras County and includes a total area of approximately 115 square miles. The uppermost portion of the watershed achieves maximum elevations of 1,000 feet and is not subject to snowmelt. It then descends through moderate slopes to the lower portion of the watershed at sea-level. The HEC-HMS model described in this memorandum has an outlet on Bear Creek at Disappointment Slough and includes Bear Creek, Upper Mosher Creek, Paddy Creek and Pixley Slough. See figure 3-12 for subbasins and index points.

4.2. Mosher Slough HEC-HMS Modeling General

Mosher Slough is located near the city of Stockton in San Joaquin County, California (Figure 2-1). The majority of the watershed is located in the urbanized area of Stockton between Interstate-5 and Highway 99 with the watershed area totaling approximately 16 square miles. The watershed's terrain has moderate slopes and reaches a maximum elevation of 65 feet above the modeled outlet at the confluence of Mosher Slough and Bear Creek just west of Interstate-5.

The HEC-HMS model described in this report includes only the lower portion of Mosher Slough which begins immediately below the diversion that routes the entirety of Upper Mosher Creek to Bear Creek (see plate 31, Figure 4- 2). The hydrology for Upper Mosher Creek is included in the Bear Creek HEC-HMS model as described in Section 3.0 of the LSJRFS Hydrology Report. See plate 32 (figure 4-10) for subbasins and index points.

4.3 Calaveras River HEC-HMS Modeling General

The Calaveras River watershed is located near the city of Stockton in San Joaquin County, California (Plates 33 and 34, Figure 5-2 and 5-12). The watershed runs east from the city of Stockton into the Sierra Nevada foothills in Calaveras County. The Calaveras River watershed can be split into two sections: above New Hogan Dam and below New Hogan Dam. The PBI - F3 Hydrology Appendix [4] focuses on the section of the Calaveras River below the dam whereas the section above the dam is part of a separate reservoir operations study [6].

The watershed includes a total area of 597 square miles with 352 square miles of this tributary area flowing into New Hogan Reservoir. The watershed discussed in this TM (below New Hogan Reservoir) includes the remaining 245 square miles and achieves maximum elevations of 1,500 feet. It then descends through moderate slopes to the lower portion of the watershed which lies at sea-level. Flow in the stream system is largely affected by releases from New Hogan Reservoir. The entire watershed is low enough in elevation to be rainfall dominant. The HEC-HMS model described in this memorandum includes the Calaveras River, Cosgrove Creek, Mormon Slough, Potter Creek, and the Stockton Diverting Canal systems and discharges to the San Joaquin River to the west of Interstate-5. See plate 34 (figure 5-12) for subbasins and index points.

4.3.1. General Characteristics of the Calaveras River Basin

The area associated with operation of the New Hogan Lake Project is basically the entire Calaveras River Basin, including its distributary channels, flood plain, and service area. The following information is taken from the New Hogan Water Control Manual, USACE, 1983).

The Calaveras River Basin above New Hogan Dam is relatively low-lying, consisting of 363 square miles on the western slope of the Sierra Nevada in Calaveras County, California. The basin is fan-shaped in plan, with the principal tributaries. Esparanza Creek and Jesus Maria Creek, which together form the North Fork of the Calaveras; and Calaveritas Creek, San Antonio Creek, and San Domingo Creek which form the South Fork. The North and South Forks join about 7 miles above the dam, within the limits of the reservoir.

Below New Hogan Dam, the Calaveras flows westerly to emerge from the foothills at Bellota, where the channel divides into two branches. A control structure provides for diversion of water when desired into the old Calaveras River channel, which is narrow and overgrown with dense vegetation. Otherwise flows enter Mormon Slough which was enlarged in the late 1960's to convey 12,500 cubic feet per second. Mormon Slough extends 13 miles southwesterly across the valley floor to the Stockton Diverting Canal, which continues northerly on the east side of Stockton to rejoin the Calaveras channel. From there, the Calaveras extends westerly through the City of Stockton to the San Joaquin River on the west side of Stockton. A General Map of the basin is presented on Plate 5 (reference plate 2) and plate 33 (figure 5-2).

4.3.2. Climate

Climate in the Calaveras River basin is characterized by cool, wet winters and hot, dry summers. Temperatures on the valley floor normally range from a winter low of about 30°F to a summer high of about 105°F and are typical of the entire basin except for the extreme upper elevations.

Normal annual precipitation (NAP) for the watershed above New Hogan Dam is 33.3 inches, and ranges from about 24 inches at New Hogan Dam to nearly 50 inches in the upper basin. In dry years, annual basin precipitation can amount to less than 11 inches and in wet years more than 40 inches. Plate 22 (reference plate 12) shows isohyetal lines of NAP over the basin.

More than 90 percent of the annual precipitation occurs from November through April. Winter storms, which account for the greatest share of annual basin precipitation, originate over the Pacific Ocean and are associated with frontal systems containing masses of moist air mov1ng inland against mountain barriers. Precipitation usually occurs as rain below 4,000 feet elevation. Above 4,000 feet, precipitation may occur as snow, although winter storms often bring rain above 4,000 feet. Intensities are moderate, but rain generally continues for three or four days and is often followed by additional storm fronts. As much as half of the normal annual precipitation may fall in a single storm period.

Precipitation during summer is from thunderstorms and is mainly confined to relatively small areas at higher elevations.

Average monthly precipitation for three representative stations are shown on Table 3.

Table 3. Precipitation Data at Selected Stations

	Average Monthly Precipitation					
	Stockton WSO Airport		Camp Pardee		Calaveras Big Trees	
Month	**Inches**	**%**	**Inches**	**%**	**Inches**	**%**
July	0.01	0.1%	0.01	0.0%	0.06	0.1%
August	0.03	0.2%	0.04	0.2%	0.13	0.2%
September	0.17	1.2%	0.18	0.9%	0.51	0.9%
October	0.72	5.1%	1.15	5.5%	2.78	5.0%
November	1.72	12.1%	2.80	13.4%	6.79	12.3%
December	2.68	18.9%	3.50	16.8%	10.17	18.4%
January	2.91	20.5%	3.85	18.5%	10.60	19.1%
February	2.11	14.9%	2.91	14.0%	8.24	14.9%
March	1.96	13.8%	3.17	15.2%	7.99	14.4%
April	1.37	9.7%	2.25	10.8%	5.25	9.5%
May	0.42	3.0%	0.80	3.8%	2.22	4.0%
June	0.07	0.5%	0.20	1.0%	0.64	1.2%
Total	14.17	100.0%	20.86	100.0%	55.38	100.0%
Nov - Apr	12.75	90.0%	18.48	88.6%	49.04	88.6%
Years of Record	27		49		35	
Elevation (feet, msl)	22		658		4695	
Basin Mean NAP	33.0 inches					
Source: NOAA NWS 1941-70						

5.0. FRENCH CAMP SLOUGH HEC-HMS MODELING GENERAL

The French Camp Slough watershed is located near the city of Stockton in San Joaquin County, California (Plates 35 and 36, Figure 6-1 and 6-2). The watershed runs east from the city of Stockton into the Sierra Nevada foothills in Calaveras County. It achieves maximum elevations of 2,100 feet and includes a total area of 430 square miles. It then descends through moderate slopes to the
lower portion of the watershed which lies at sea-level. None of the watershed experiences snowfall; all floods are rainfall-induced.

The HEC-HMS model described in this memorandum includes the Duck Creek, Lone Tree Creek, Temple Creek, Rock Creek, Webb Creek, Littlejohn Creek, and the French Camp Slough systems and discharges to the San Joaquin River to the west of Interstate-5. See plate 36 (figure 6-11) for subbasins and index points.

5.1. Littlejohn Creek Watershed Characteristics

The following information is taken from the Farmington Dam Water Control Manual, USACE, 2004.

5.1.1 General Characteristics.

The basin encompassing the Littlejohn Creek Stream Group – bounded on the north and south by the Calaveras and Stanislaus river basins, respectively – is about 15 miles (24.1 km) wide from north to south and 40 miles (64.4 km) long from east to west. Runoff from its approximately 415 square mile drainage area flows westward to the San Joaquin River via French Camp Slough. Of the many creeks comprising the Littlejohn Creek Stream Group, three are considered major: Littlejohn, Duck, and Lone Tree, and of these, Littlejohn is the principal stream system.

Above Farmington Dam, the watershed portion of the project is a wing-shaped area extending 20 miles (32.0 km) upstream into the foothills on the western slope of the Sierra Nevada. Principal streams contributing to the reservoir are Littlejohn, Rock and Hoods creeks. These streams drain a combined area of 212 square miles at the dam. Above the diversion structure, across Duck Creek, the drainage area is 28 square miles. Basin features are shown on the General Map, plates 28, 35 and 36 (figures 2-1, 6-2 and 6-11).

Vegetative cover varies within the basin. Above Farmington Dam, the steep hillsides in the upper basin are sparsely covered by deciduous brush, small stands of trees, and a grassland understory. A discontinuous bank of riparian growth stretches through much of the upper basin. Along portions of Rock and Littlejohn creeks, the banks are completely devoid of riparian vegetation and badly eroded. The existing riparian vegetation is primarily valley oak, Fremont cottonwood, willow and white alder. Shrubs include willow, elderberry, and coyote brush. Annual grassland, such as grasses and forbs, is the predominant vegetation type within the reservoir area. Below Farmington Dam, the lower basin consists primarily of intensely developed agricultural lands and unimproved pastureland. Along lower basin stream channels, native vegetation has diminished, with some light brush and a few scattered oaks remaining.

5.1.2. Climate

a. General. The climate of the Littlejohn Creek Basin is classified as dry and sub-humid, characterized by two well-defined seasons: long, hot dry summers with very little rain, and short, mild wet winters with frequent rain but very little snow. The location of climatological stations and normal annual precipitation isohyets are shown on plates 24 and 26 (Plate 4-5.1 and 4-5.2).

b. Temperature. Average temperatures within the basin range between 45°F and 77°F, with a yearly average of 61.5°F. Summer highs can reach 115°F and winter lows can drop to near freezing. At Stockton, extreme temperatures have ranged from 114°F during the summer to 16°F during the winter months.

c. Precipitation. Normal annual precipitation (NAP) varies throughout the Littlejohn Creek drainage area, ranging from 12 inches on the valley floor to about 30 inches in the higher areas as shown on plates 24 and 26 (Plate 4-5.1 and 4-5.2). Normal annual precipitation above Farmington Dam is about 17 inches, while downstream it is about 14 inches. The mean monthly and annual distribution of precipitation at selected stations is given in Table 4.

TABLE 4

MEAN MONTHLY PRECIPITATION

MONTH	STOCKTON WSO AIRPORT + (Elev 22')		KNIGHTS FERRY 2ESE ‡ (Elev 315')		COPPEROPOLIS ‡ (Elev 970')		FLOWERS MOUNTAIN (Elev 1480')	
	in	%	in	%	in	%	in	%
Jan	2.85	20.4	2.88	16.9	4.52	19.4	4.07	19.2
Feb	2.27	16.3	2.55	15.0	4.08	17.6	3.99	18.8
Mar	2.04	14.6	2.49	14.6	3.83	16.5	3.51	16.5
Apr	1.13	8.1	1.74	10.2	1.80	7.7	1.60	7.5
May	0.41	2.9	0.39	2.3	0.46	2.0	0.82	3.9
Jun	0.08	0.6	0.15	0.9	0.19	0.8	0.21	1.0
Jul	0.03	0.2	0.10	0.6	0.06	0.3	0.09	0.4
Aug	0.04	0.3	0.15	0.9	0.08	0.3	0.08	0.4
Sep	0.28	2.0	0.29	1.7	0.31	1.3	0.18	0.9
Oct	0.69	5.0	0.96	5.6	1.06	4.6	1.29	6.1
Nov	1.81	13.0	2.65	15.5	3.20	13.8	2.53	11.9
Dec	2.31	16.6	2.69	15.8	3.66	15.7	2.85	13.4
Average Annual	13.94	100.0	17.04	100.0	23.25	100.0	21.22	100.0
Nov-Mar	11.28	80.9	13.26	77.8	19.29	83.0	16.95	79.5
Source:	NOAA 1941-2004		NOAA 1960-1972 1974-1976		USACE 1955-1995		USACE 1972-2003	

+ Climatological Data Summary. Monthly Average Temperatures (updated June 2004) retrieved 12 July 2004 from Western Regional Climate Center, Desert Research Institute Web site: <http://www.wrcc.dri.edu/> ‡Gage discontinued.

About 80 percent of the precipitation runoff occurs during the months of November through March. Snow rarely falls on the area and is not a significant factor in runoff from large storms.

6.0. DESIGN STORMS

Except for Bear Creek (storm balanced to multiple durations), design storms for hydrologic analysis of the Mosher Slough, Calaveras River below Bellota, and Littlejohn and French Camp system below the town of Farmington were created using 72-hour duration NOAA14 depths and areal reduction for the 1/2, 1/5, 1/10, 1/25, 1/50, 1/100, 1/200, and 1/500 AEP events as input to the LSJRFS HEC-HMS models. As discussed in Section 6.3, the 72-hour storm pattern provides a storm event that is high in both peak flow and volume which is important for levee breach scenarios.

6.1. Rainfall Zones

LSJRFS subbasins were aggregated into seven rainfall zones with uniform rainfall characteristics. Seven rainfall gages were selected to form the basis of this subbasin aggregation. The selected gages are distributed throughout the study area and have available rainfall data at short-interval timesteps which can be used for storm patterning (see Section 6.3).

GIS software was used to draw Thiessen polygons around the selected rainfall gages and subbasins lying within each Thiessen polygon were aggregated to create the rainfall zones Plate 28 (Plate 2-1).

6.2. Design Storm Depths

The National Oceanic and Atmospheric Administration (NOAA) published its Atlas 14 Precipitation Frequency Study for California1 in April 2011 (NOAA, 2011) which includes estimates for design rainfall depths in an ASCII grid file format for use in GIS. A shapefile with seven defined rainfall zone boundaries was projected on top of the NOAA14 ASCII grid files to calculate average point rainfall depths within each rainfall zone for 96 different frequency-duration combinations.

The output from the NOAA14 GIS data acquisition process includes depth-duration-frequency tables for each rainfall zone. These depth-duration-frequency tables are included for each watershed in their respective attachments.

6.3. Design Storm Pattern

The design storm pattern used for the LSJRFS is based on an observed storm event that was recorded at various rainfall gages within the study area.

The December 31, 1996-January 3, 1997 rainfall event (1997 Event) and the April 2, 2006-April 5, 2006 rainfall event (2006 Event) were considered for the basis of design storm patterning. These events represent two of the largest storms in recent history.

Data records were checked for these events at all known precipitation gages within the vicinity of the study area. Some gages only had recorded data at monthly or daily intervals and were excluded from the gage selection process based on their inadequate time step. Other gages were excluded due to lack of data for the specific dates listed; many of the available rainfall gages did not contain data for the 2006 Event.

The 1997 Event is often considered an industry standard for rainfall events and was ultimately selected as the pattern used to temporally distribute the design storms. The storm temporal pattern is shown below in figure 5.1.

Data from the New Hogan (NHG) gage location represents a typical 72-hour hyetograph pattern for the 1997 Event and is shown below.

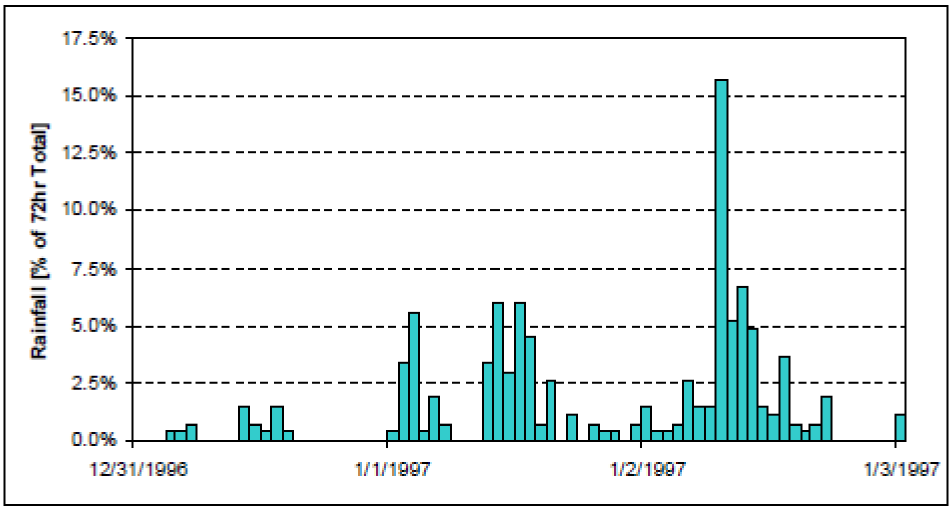

Figure 5.1. Typical Rainfall Pattern for the 1997 Event.

The 72-hour storm pattern provides a storm event that is high in volume which is important for levee breach scenarios. For the LSJRFS, it is also desirable to preserve the high peak flows that would result from a standard, 24-hour design storm. Therefore, additional analyses were conducted for Mosher and French Camp Sloughs to run a SCS Type 1 storm, an industry standard 24-hour event, to confirm that the peak flows resulting from either type storm were comparable. For the lower Calaveras River watershed below Bellota, a 97 pattern balanced to 1-, 3-, 6-, 12-, 24-, 48-, and 72 hour NOAA14 depths and areal reduction factors was compared to the 97 pattern balanced only to a 72-hour depth and one areal reduction factor. The results were highly comparable in volume and peak (see Appendix 2).

All flows were comparable except for those in the Bear Creek watershed. To correct this, Bear Creek hyetographs were balanced to the 3-, 6-, 12-, 24-, 48-, and 72-hour NOAA14 storm depths. After balancing the hyetographs, Bear Creek models produced high-volume hydrographs with peak flows that are comparable to those resulting from a standard 24-hour design storm.

6.4. Storm Centering Approach

The LSJRFS utilizes a storm centering approach to consider depth area reduction of design storms falling over the study area. This area reduction is typically disregarded for small watersheds where one point precipitation depth can be applied to the entire tributary area, however given the size of the watersheds in the LSJRFS it is necessary to apply area reduction factors to the point rainfall design storm depths.

Area reduction factors were calculated using a procedure that was developed by the USACE Sacramento District for the hydrology of their Downtown Guadalupe River Project in November 2009 [9]. This procedure takes into account various storm centerings by ranking the rainfall zones according to their distance from the storm centering location and determining the cumulative drainage area for each location in the watershed. HMR 59 was source of factors.

7. EXISTING CONDITIONS

Existing conditions are those at the time the study is conducted and form the basis for extrapolations to other conditions. Existing conditions within the study area are discussed below.

7.1 Flow Frequency Estimates

Flood waters potentially threatening the study area originate from several sources.

Those sources include:

- The San Joaquin river mainstem (flood control projects are shown in table 10 below);
- The east side tributaries including:
    - Bear Creek,
    - Mosher Slough,
    - Calaveras River and Mormon Slough,
    - Littlejohn Creek, Duck Creek, and French Camp Slough;
- The Sacramento-San Joaquin Delta, including the Sacramento, San Joaquin, Cosumnes and Mokelumne Rivers, and ocean tides.

The discharges by index point for annual exceedance probabilities of 0.5 (1/2) to 0.002 (1/500) are shown in table 5 below. Plates 30, 32, 34, and 36 (figures 3-12, 4-10, 5-12, and 6-11), at the end of this memo, show the location of the index points.

The existing and future without project conditions are considered the same. In addition, the future with project condition is essentially the same as the existing without project condition. Therefore, the table of existing conditions flow values will be used for all conditions.

Table 5. Existing Conditions Regulated Flows (CFS)

Existing Conditions Regulated Discharge Summary Table at Index Points											
				Regulated Peak Discharge or Stage by Return Period and Annual Exceedance Probability							
Stream	Index Point	Drainage Area	Period of Record (years)	2 0.5	5 0.2	10 0.1	25 0.04	50 0.02	100 0.01	200 0.005	500 0.002
Bear Creek	Lockeford gage	47.6	51								
Bear Creek	BL4	79.5	25	1,900	2,680	3,300	4,180	4,890	5,560	6,320	7,410
Bear Creek	BL3	91.9	25	2,060	2,940	3,630	4,810	5,710	6,620	7,570	8,880
Bear Creek	BR4	94.2	25	2,060	2,940	3,670	4,850	5,770	6,680	7,650	8,970
Bear Creek	BR3	95.1	25	2,060	2,940	3,690	4,870	5,790	6,700	7,670	9,000
Bear Creek	BL2	95.9	25	2,050	2,940	3,700	4,900	5,810	6,730	7,700	9,030
Bear Creek	BL1	97.3	25	2,050	2,950	3,740	4,950	5,870	6,800	7,780	9,110
Bear Creek	BR2	99.0	25	2,050	2,960	3,790	5,020	5,940	6,880	7,870	9,210
Bear Creek	BR1	114.2	25	2,080	2,990	3,840	5,180	6,200	7,240	8,340	9,820
Bear Creek	D2	-	57	7.25	8.20	8.90	9.05	9.29	9.45	9.58	9.76
Mosher Slough	ML2	1.28	20	170	230	230	230	230	230	230	510
Mosher Slough	ML1	7.55	20	440	620	690	800	890	940	960	970
Calaveras River	New Hogan Dam	363	86	3,320	8,990	9,310	10,440	12,330	12,400	12,500	14,820
Cosgrove Creek	Valley Springs	21.1	51	1,020	1,880	2,480	3,240	3,780	4,310	4,810	5,440
Calaveras River	Bellota	470	104	3,520	9,520	9,530	10,640	12,500	12,500	12,500	16,000
Calaveras River	CL3	26.4	95	110	230	300	440	530	620	720	810
Calaveras River	SL2	488	95	4,150	10,150	10,620	12,140	14,210	14,960	15,320	19,510
Calaveras River	SR1	503	95	4,150	10,150	10,630	12,150	14,220	14,970	15,340	19,530
Calaveras River	SL1	532	95	4,150	10,150	10,670	12,230	14,320	15,070	15,440	19,620
Calaveras River	CR2 & CL2	591	95	3,810	9,620	10,050	12,530	13,670	15,650	16,110	20,230
Calaveras River	CR1	594	90	3,700	9,660	9,780	12,520	13,320	15,610	16,100	20,190
Calaveras River	CL1	594	90	3,700	9,660	9,780	12,520	13,320	15,610	16,100	20,190
Calaveras River	D4 & D5	-	57	7.30	8.30	8.95	9.20	9.30	9.40	9.49	9.60
Littlejohn Creek	Farmington Dam	212	53	1,400	2,170	2,370	1,990	3,360	8,660	12,000	16,210
Littlejohn Creek	at Farmington	247.9	58	1,400	2,170	2,370	2,620	3,740	9,900	12,900	16,600
Duck Creek	Farmington	8.25	58	130	200	240	300	340	380	420	470
French Camp Slough	FL1, FR1	-	57	7.30	8.30	8.95	9.20	9.30	9.40	9.49	9.60
San Joaquin River	Vernalis	13,536	82	25,000	32,000	35,109	42,000	47,676	78,209	124,587	165,208

Notes:
Bear Creek, Mosher Slough, Cosgrove Creek, and Duck Creek are unregulated streams.
The discharge values in this table represent the worst case storm centering.
The index point locations are shown on plates 1 to 4.
See the Hydrology Appendices by Ford or PBI for details not shown here.
Bear Creek index point D2, Calaveras index points D4 & D5, and French Camp index points FL1 & FR1 are based on a tide stage frequency analysis.
The flows for the San Joaquin river were extracted from a UNET model from the Comp Study 2002.

Table 6. Future Conditions Regulated Flows (CFS)

Future Conditions Regulated Discharge Summary Table at Index Points

Stream	Index Point	Drainage Area	Period of Record	Regulated Peak Discharge or Stage by Return Period and Annual Exceedance Probability							
				2 0.5	5 0.2	10 0.1	25 0.04	50 0.02	100 0.01	200 0.005	500 0.002
Bear Creek	Lockeford gage	47.6	51								
Bear Creek	BL4	79.5	25	1,900	2,680	3,300	4,180	4,890	5,560	6,320	7,410
Bear Creek	BL3	91.9	25	2,060	2,940	3,630	4,810	5,710	6,620	7,570	8,880
Bear Creek	BR4	94.2	25	2,060	2,940	3,670	4,850	5,770	6,680	7,650	8,970
Bear Creek	BR3	95.1	25	2,070	2,960	3,710	4,890	5,820	6,730	7,700	9,010
Bear Creek	BL2	95.9	25	2,070	2,970	3,740	4,920	5,860	6,790	7,790	9,100
Bear Creek	BL1	97.3	25	2,080	2,980	3,790	5,000	5,920	6,900	7,870	9,230
Bear Creek	BR2	99.0	25	2,110	3,020	3,840	5,050	6,070	7,030	7,960	9,380
Bear Creek	BR1	114.2	25	2,170	3,070	4,050	5,470	6,600	7,750	8,810	10,410
Bear Creek	D2	-	57	7.25	8.20	8.90	9.05	9.29	9.45	9.58	9.76
Mosher Slough	ML2	1.28	20	170	230	230	230	230	230	230	510
Mosher Slough	ML1	7.55	20	440	620	690	800	890	940	960	970
Calaveras River	New Hogan Dam	363	86	3,320	8,990	9,310	10,440	12,330	12,400	12,500	14,820
Cosgrove Creek	Valley Springs	21.1	51	1,020	1,880	2,480	3,240	3,780	4,310	4,810	5,440
Calaveras River	Bellota	470	104	3,520	9,520	9,530	10,640	12,500	12,500	12,500	16,000
Calaveras River	CL2	26.4	95	110	230	300	440	530	620	720	810
Calaveras River	SL1	488	95	4,150	10,150	10,620	12,140	14,210	14,960	15,320	19,510
Calaveras River	SR0	503	95	4,150	10,150	10,630	12,150	14,220	14,970	15,340	19,530
Calaveras River	SL0	532	95	4,150	10,150	10,670	12,230	14,320	15,070	15,440	19,620
Calaveras River	CR2 & CL1	591	95	3,810	9,620	10,050	12,530	13,670	15,650	16,110	20,230
Calaveras River	CR0	594	95	3,700	9,660	9,780	12,520	13,320	15,610	16,100	20,190
Calaveras River	CL0	594	95	3,700	9,660	9,780	12,520	13,320	15,610	16,100	20,190
Calaveras River	D4 & D4	-	57	7.30	8.30	8.95	9.20	9.30	9.40	9.49	9.60
Littlejohn Creek	Farmington Dam	212	53	1,400	2,170	2,370	1,990	3,360	8,660	12,000	16,210
Littlejohn Creek	at Farmington	247.9	58	1,400	2,170	2,370	2,620	3,740	9,900	12,900	16,600
Duck Creek	Farmington	8.25	58	130	200	240	300	340	380	420	470
French Camp Slough	D7 & D7	-	57	7.30	8.30	8.95	9.20	9.30	9.40	9.49	9.60
San Joaquin River	Vernalis	13,536	82	25,000	32,000	35,109	42,000	47,676	78,209	124,587	165,208

Notes:
Bear Creek, Mosher Slough, Cosgrove Creek, and Duck Creek are unregulated streams.
The discharge values in this table represent the worst case storm centering.
The index point locations are shown on plates 1 to 4.
See the Hydrology Appendices by Ford or PBI for details not shown here.
Bear Creek index point D2, Calaveras index points D4 & D5, and French Camp index points FL1 & FR1 are based on a tide stage frequency analysis.
The flows for the San Joaquin river were extracted from a UNET model from the Comp Study 2002.

Table 7. Existing Conditions Unregulated Flows (CFS)

Existing Conditions Unregulated Discharge Summary Table at Index Points											
		Drainage Area	Period of Record	Unregulated 1-day Discharge by Return Period and Annual Exceedance Probability							
				2	5	10	25	50	100	200	500
Stream	Location	(sq mi)	(years)	0.5	0.2	0.1	0.04	0.02	0.01	0.005	0.002
San Joaquin River	Maze Road		82	19,203	44,753	68,988	108,667	145,171	187,885	237,393	314,324
San Joaquin River	Vernalis	13,536	82	24,126	56,984	88,444	140,317	188,312	244,715	310,343	412,740
Littlejohn Creek	Farmington Dam	212	58	2,471	5,682	8,061	11,034	13,118	15,044	16,810	18,903
Littlejohn Creek	at Farmington	247.9	58	2,730	7,015	10,438	14,930	18,192	21,282	24,173	27,668
Duck Creek	Farmington	8.25	58	128	196	241	297	339	379	419	472
Calaveras River	New Hogan Dam	363	104	5,627	13,000	18,618	25,855	31,081	36,039	40,701	46,391
Cosgrove Creek	Valley Springs	21.1	51	339	614	804	1,039	1,208	1,369	1,523	1,716
Calaveras River	Bellota	470	104	6,909	15,401	21,677	29,582	35,185	40,426	45,293	51,153

Notes:
The discharge values in this table represent the worst case storm centering.
The index point locations are shown on plate 5.
See the Calaveras River and Littlejohn Creek Frequency Reports by David Ford Consulting Engineers for details on those streams.
See the Sacramento-San Joaquin Comprehensive Study for details on the San Joaquin River.

Flow frequency estimates for the San Joaquin River are based on analysis described in the Sacramento and San Joaquin River Basins Comprehensive Study documentation. Flow frequency curves and hydrographs of unregulated flow were developed for the 50% (1/2) to 0.2% (1/500) Annual Chance Exceedance probability (ACE) frequencies. Regional synthetic hydrology presented in these studies represents the best available data for the large flood sources (San Joaquin River) of the Lower San Joaquin River Feasibility Study. These hydrologic analyses have also been used as the foundation for several other feasibility studies in the region, such as the Sutter Basin Feasibility Study. DWR and USACE are in the process of developing new hydrologic frequency estimates for existing conditions; however, the results are not available until mid-2014. Therefore, this study utilizes the results from the Sacramento and San Joaquin River Basins Comprehensive Study hydrologic analysis.

Synthetic hydrology of the Sacramento and San Joaquin River Basins Comprehensive Study was based on transformation of unregulated hydrologic conditions to regulated conditions. This was accomplished by developing balanced unregulated hydrographs based upon historically patterned storm events. Balanced hydrographs have the same annual exceedance frequency for all flood durations. For example a 10% (1/10) ACE hydrograph contains the 10% (1/10) ACE 1-day flow, 10% (1/10) ACE 3-day average flow, 10% (1/10) ACE 5-day average flow etc. These balanced hydrographs were then transformed to regulated hydrographs using an HEC-5 reservoir operations model of the system. The HEC-5 model, also developed and calibrated for the Sacramento and San Joaquin River Basins Comprehensive Study, simulates reservoir operations and produces regulated hydrographs. The comprehensive study transferred the hydrographs from the HEC-5 model at 'handoff' points and modeled in more hydraulic detail using UNET. The portion of the UNET model downstream of the San Joaquin River at Newman was replaced by an HEC-RAS unsteady model developed for this study (see hydraulics section). Hydrographs at San Joaquin River at Newman were obtained from the UNET model. All other hydrograph boundary conditions were obtained from the HEC-5 model. This process is shown on plate 19 (reference plate 6).

The Sacramento and San Joaquin River Basins Comprehensive Study hydrology utilized a runoff centering approach to evaluate possible hydrologic scenarios. A centering is multiple and varying frequency hydrographs positioned (centered) over a watershed to produce flow rates or stages of one specific frequency at a specific location (like Vernalis). Multiple centering scenarios are possible due to the diverse spectrum of floods that can occur from different combinations of concurrent storms on tributaries, orographic influences, and other factors that influence regional rainfall runoff events. The Comprehensive Study evaluated a suite of recorded flood centerings and generally tried to mimic general characteristics of those that historically produced the higher flows at a given location. For the Lower San Joaquin Feasibility study area, the Sacramento and San Joaquin River Basins Comprehensive Study results were reviewed and narrowed to one possible centering. The San Joaquin at Vernalis storm centering predominantly applies to the San Joaquin River downstream of Vernalis and the Stockton area.

7.2 Risk and Uncertainty Parameters

Uncertainties that Most Influence the Alternative Selection

For this study, Corps risk assessment procedures, incorporating uncertainty analysis, were followed. These procedures incorporate the best-available hydrologic, hydraulic, geotechnical, and economic information to compute expected annual damage (EAD), accounting explicitly for uncertainty in the information.

Each aspect of the flood risk assessment must account for uncertainty. For hydrologic and hydraulic analysis, the principle variables are discharge and water surface elevation. Uncertainty in discharge exists because record lengths are often short or do not exist where needed, precipitation-runoff computation methods are inaccurate, and the effectiveness of flood flow regulation measures is not known precisely. Uncertainty factors that affect water surface elevation include conveyance roughness, cross-section geometry, debris accumulation, ice effects, sediment transport, flow regime, and bed form. For geotechnical and structural analyses, the principle source of uncertainty is the structural performance of an existing levee due to its physical characteristics and construction quality. Uncertainty also arises from a lack of information about the relationship between depth and inundation damage, lack of accuracy in estimating structure and content values and locations, and the lack of ability to predict how the public will respond to a flood. These specific variables were explicitly accounted for in this risk assessment and via a sensitivity analysis the uncertainty in the hydrology most influence the damage and engineering performance outputs and thus the alternative selection. However, variables not explicitly evaluated that could influence future performance include climate change, or unforeseen changes in the watershed conditions such as unplanned growth or dramatic changes in agricultural practices.

Risk is defined as the probability that an event will occur, and the consequence of that outcome. Uncertainty is defined as a measure of insufficient knowledge of parameters and functions used to describe the hydraulic, hydrologic, geotechnical and economic aspects of a project plan. Risk analysis is an approach to evaluation and decision-making that explicitly incorporates estimates of risk and uncertainty in a flood damage reduction study. The annual exceedance probability or AEP is the probability that a flood event will occur in any given year, considering the full range of possible annual floods.

Unregulated flow frequency curves for Mormon Slough at Bellota, Farmington Dam, Littlejohn Creek at Farmington, and the San Joaquin River at Vernalis were developed by the direct analytical approach. A reservoir routing model was then used to regulate unregulated hydrographs. The direct analytical approach is used when a sample of stream gauge annual discharge values are available and the data can be fit with a statistical distribution. The median function is used in the risk based analysis. The derived function may then be used to predict specified exceedance probabilities. The approach generally follows USACE guidance including EM 1110-2-1415 and ER 1110-2-1450. The confidence limits will be computed within the HEC-FDA program from the period-of-record provided with the flow frequency statistics. An unregulated to regulated transform will be linked with the unregulated flow frequency curve in FDA. The lower Calaveras River watershed downstream of Mormon Slough at Bellota was modeled using a rainfall runoff model to produce concurrent local flow runoff when an a specific frequency event occurs at Bellota. Since approximately 75% or more of the total flow contained in the watershed's levees comes from sources upstream of Bellota, a decision was made to use the unregulated 1-day frequency curve statistics with equivalent period of record for all downstream index points (except those impacted by Delta tides). An unregulated to regulated peak flow transform is linked to the unregulated 1-day frequency curve in FDA, with regulated peak based on the peak of the various frequency rainfall runoff model hydrographs produced at each index location.

The flood flow frequency estimates for Bear Creek, Mosher Slough, and for French Camp Slough downstream of Littlejohn Creek at Farmington were developed as hypothetical frequency events in a rainfall runoff model. In this case unique discharge hydrographs due to storms of specified probabilities and temporal and areal distributions are computed with a rainfall-runoff model. Flow frequency curves from rainfall runoff models are typically expressed as a graphical function. The graphical approach uses plotting positions to define the relationship with the actual function fitted by "eye" through the plotting position points. The confidence limits for flood flow estimates developed by use of rainfall-runoff models will be by equivalent record length guidelines as shown in table 8 below. Table 8 was extracted from EM 1110-2-1619, table 4-5.

Delta gage stage frequency curves and associated periods of record were used for tidally influenced points on the lower Bear Creek, lower Calaveras River, and French Camp Slough.

The final assessment of equivalent record length for each location is presented in tables 5 and 6.

TABLE 8

Equivalent Record Length Guidelines	
Method of Frequency Function Estimation	**Equivalent Record Length[1]**
Analytical distribution fitted with long-period gauged record available at site	Systematic record length
Estimated from analytical distribution fitted for long-period gauge on the same stream, with upstream drainage area within 20% of that of point of interest	90% to 100% of record length of gauged location
Estimated from analytical distribution fitted for long-period gauge within same watershed	50% to 90% of record length
Estimated with regional discharge-probability function parameters	Average length of record used in regional study
Estimated with rainfall-runoff-routing model calibrated to several events recorded at short-interval event gauge in watershed	20 to 30 years
Estimated with rainfall-runoff-routing model with regional model parameters (no rainfall-runoff-routing model calibration)	10 to 30 years
Estimated with rainfall-runoff-routing model with handbook or textbook model parameters	10 to 15 years

[1] Based on judgment to account for the quality of any data used in the analysis, for the degree of confidence in models, and for previous experience with similar studies.

This table was developed after table 4-5 in EM 1110-2-1619, Risk based analysis for flood damage reduction studies.

Bear Creek hydrology is based on a rainfall-runoff model calibrated to an observed event at a short-interval runoff gage.

Mosher Slough is based on a rainfall runoff model. The model wasn't calibrated to an observed event, however, because stream flows are largely dependent on pumped flows, the degree of uncertainty is judged to be equivalent to a calibrated model.

The Mormon Slough at Bellota index point equivalent record is based on "half" the period of record of the 1-day unregulated flow frequency curve at that location. It was reduced in half because of uncertainty about how efficiently the dam can operate to local flow conditions. This equivalent record was also adopted for multiple index points downstream of Bellota since approximately 75% or more of the total flow in the downstream levees is from sources upstream of Bellota.

The equivalent record length for French Camp Slough is based on the period of record of the tide gages analyzed for this location. Backwater from the San Joaquin River and the Delta (not discharges from the French Camp Slough watershed) determine the highest stages at this location. Littlejohn Creek at Farmington equivalent record is based on the period of record of the unregulated flow frequency curves at that location. There were no gages to calibrate the Duck Creek portion of the rainfall runoff model. The entire French Camp Slough rainfall runoff model (used to produce concurrent local flow contributions downstream of Littlejohn Creek at Farmington, Ca including Duck Creek) wasn't calibrated to an observed event; however the soil loss rates were adjusted based on the calibration of the neighboring Calaveras River model.

The equivalent period of records that are used in HEC-FDA to establish the confidence limits for the flood flow frequencies are shown in tables 5 and 6.

8.0 FLOOD DAMAGES

Major flooding occurred in San Joaquin, Stanislaus, and Merced counties along the lower San Joaquin River in 1983, 1986, 1995 and 1997 [10]. The distribution of flood damages among the three counties has varied considerably depending upon storm paths. However, the highest magnitude of damages occurred to agricultural crops and developments. The 1997 flood event did, however, damage 1,842 residences, mobile homes, and businesses in San Joaquin and Stanislaus counties. Estimated average annual equivalent damages (year 2000) from floods in the Lower San Joaquin River Basin amount to about $20 million based on preliminary HEC-FDA model for the Comprehensive Study. Crop damages ($9 million) account for nearly half of the estimated damages.

Table 11 below entitled “Historical Flooding in the Calaveras River” is provided using data from the 1983 Water Control manual and updated through 2012 with data from CDEC and Corps files.

There is some evidence to suggest that sediment deposition has contributed to reducing channel capacities and contributed to flood problems within the study area. Past farming practices directed sediment-laden agricultural drainage from fields to the river. Current practices are attempting to retain agricultural drainage on site. Upstream diversions on the San Joaquin River and tributaries have reduced the frequency of high flows, thereby reducing the transport of sediment through the river system.

The portion of the study area between Stockton and Tracy has experienced significant development within the past decade. The River Islands master planned community is currently proposed for 5,000 acres of the Stewart Tract between Paradise Cut, the San Joaquin River, and Old River. Applications for Corps and Central Valley Flood Protection Board (CVFPB) permits are currently pending. The proposed project would increase the conveyance capacity of Paradise Cut by setting back approximately 20,000 feet of existing levee and dry excavating approximately 3,000,000 cubic yards of material within the levee setback area. Paradise Cut is a bypass channel connecting to the San Joaquin River and increasing conveyance in the upstream portion of the San Joaquin River.

Flood damages along the San Joaquin River will likely continue to increase due to population growth and urban development. Although new structures will need to comply with land use regulations pursuant to the National Flood Insurance Program (NFIP), there will continue to be increases in flood damages due to residual risks from floods exceeding designed levels of protection, increased flood damages to automobiles and other property outside of regulated structures, and improvements to existing structures in the floodplain that increase the amount of property exposed to potential flood damages.

8.1. Storms and Floods in the Calaveras River Basin including New Hogan Dam

Rain floods can occur anytime during the period from November through April. This type of flood is usually caused by frontal systems from the Pacific Ocean moving against the Sierra Nevada. Rainfall intensities are generally moderate but prolonged over several days. The resulting floods are usually characterized by high peak flows of short duration, but when antecedent rainfall has resulted in saturated ground conditions or when the ground is frozen, the volume of runoff is much greater and flooding is more severe. [11].

Since the Calaveras River Basin is low-lying, snow and snowmelt runoff are negligible in contributing to flooding.

Thunderstorms lasting up to three hours can occur over small areas at higher elevations from late spring through early fall. The resulting runoff is characterized by high peak flows of short duration with low volumes. For small tributaries, peak flows from thunderstorms can approach those which occur during major winter rain floods, but flows on the Calaveras River are barely affected.

Quantitative information on flooding in the study area prior to 1900 is practically non-existent. Streamflow records extend from 1901 to the present for the Calaveras River. Descriptive data on flood events since the turn of the century may also be found in newspaper files; the authorization documents for the flood control projects on the Calaveras River; certain of the design documents for these projects; publications of the U.S. Geological Survey and U.S. Weather Bureau (now National Weather Service); and, since 1950, in unpublished post-flood reports prepared by the Corps of Engineers.

Although quantitative data does not exist for historical floods, descriptions of floods in the last half of the 19th Century indicate their large magnitudes. It is recorded that valley floor area of the Calaveras River was entirely inundated during a number of these floods; during floods that occurred in 1861-62, flooding on the valley floor was deep enough to permit riverboats to reach almost any locality in the inundated area.

The major floods that occurred during the earlier part of the 20th Century (March 1901, January 1909, January-February 1911, and January 1921) were all very similar in their impacts. Flooding was widespread, frequently extending entirely across the area between Mormon Slough and the Calaveras River in the vicinity of Linden, which was entirely flooded a number of times during the period. Subsequent to construction of the Diverting Canal (1910), floodwater ponded on its north side and extended far to the north and east. The area was frequently described as an inland sea. These floods caused extensive damage and great hardship, and repair, restoration, and recovery created major financial burdens on the county government and on the individuals directly affected.

Subsequent to 1936, the original Hogan Dam and Reservoir had a tempering effect on flooding in the study area. Floods that would have reached major proportions were largely averted by that project in February 1938 and February 1963.

The most widespread and destructive flood of any in the recorded history of the Central Valley occurred in December 1955. Floodwater broke out of the Calaveras River to inundate farmlands in the vicinity of Linden. Mormon Slough breached its levees and flooded along both sides from Bellota to the Diverting Canal. An extensive area north and east of the canal was inundated.

During the 1958 flood, Hogan Reservoir filled and spilled for the first time since its completion. About 3,000 acres of farmlands in the vicinity of Linden were flooded by the Calaveras River where two levee breaks occurred. Linden was threatened but not damaged. Levees along Mormon Slough were breached in a number of locations and about 7,000 acres of land flooded in a strip extending from Bellota to the Diverting Canal. A major levee break occurred near the head of the Diverting Canal. Flooding also occurred on 1,500 acres along the north side of the Diverting Canal.

Widespread flooding occurred in northern and central California and western Nevada in December 1964 and January 1965. Severe storms occurred over the watershed but flooding and flood damage was minimal because the levee and channel improvement project was nearly finished at the time and functioned effectively to prevent significant damage to agricultural and suburban residential developments. New Hogan Dam, which became operational just prior to the flood season, stored runoff from a moderately large flood and controlled flows downstream to non-damaging amounts.

8.2. Storms and Floods in the Littlejohn Creek Basin including Farmington Dam

Littlejohn Creek Basin lies on the western, or seaward, slope of the Sierra Nevada. The basin is partially shielded from general storms by the barrier of the Coast Ranges. The peaks rise from 3,000 to 5,000 feet (914 to 1,524 m) in elevation. General rain storms are carried into the basin by moist, unstable Pacific air masses that travel through the San Francisco Bay from the northwest. The Coast Range influences the rate and duration of precipitation that falls on the Littlejohn Creek Basin. General rain floods occur primarily between November and March. Prolonged heavy rainfall produces general rain floods characterized by high peak flows of moderate duration (2-3 days) and relatively shallow depths of 2 to 3 feet (61.0 to 91.4 cm). When antecedent rain has saturated the ground, flooding is more severe. [12].

Comparative flows for observed floods in the Littlejohn watershed since the turn of the century are shown in Table 9 on the next page. It should be noted that damage in the study area during most of the known past floods would have been significantly reduced if the floods had occurred with presently existing flood control facilities completed and in operation.

TABLE 9 **HISTORICAL FLOOD FLOWS ON LITTLEJOHN CREEK AT FARMINGTON DAM**			
DATE	PEAK (cfs)	1-DAY VOL (acre-feet)	3-DAY VOL (acre-feet)
February 1986	23,600	18,952	45,593
April 1958	28,900	14,424	41,136
December 1955	20,000	16,854	34,727
February 1998	24,830	22,865	32,216
January 1983	16,500	12,986	28,128
Source: Water Management Section, Sacramento District, USACE			

Other major floods within this century occurred in January-February 1911 and February 1917. Peak flows prior to these project events were 16,000 and 13,600 cfs, respectively. The legendary floods of 1861-1862 are judged to be the largest in peak flow and volume of runoff, but were less damaging than the floods listed due to the area being less populated and developed.

Farmington Reservoir offers flood protection to about 58,000 acres of agricultural land, suburban areas, and industrial properties in the area immediately south of Stockton. Flood damages within the basin are primarily agricultural. Four of the largest floods of record occurred in December 1955, April 1958, February 1986, and February 1998. Maximum storage (53,512 acre-feet) occurred in February 1998. Peak outflow (2,438 cfs) occurred in February 1986. Peak inflow (28,900 cfs) occurred in April 1958, as did the largest flows on Duck and Littlejohn creeks. In April 1958, Duck Creek flows at the Diversion reached a peak of 4,100 cfs, compared with 2,700 cfs in February 1986, 2,600 cfs in December 1955, and 2,100 cfs in February 1998. Similarly, the flow at Farmington peaked at 3,600 cfs in April 1958, compared with 3,000 cfs in February 1986, 2,750 cfs in December 1955, and 2,400 cfs in February 1998. The 1955 and 1958 floods caused much damage.

However, no significant flooding occurred within the Littlejohn Creek basin for the February 1986 event.

In December 1955, flooding in the Littlejohn Creek area affected about 1,800 acres. Farmington Reservoir controlled Littlejohn Creek inflows to a safe channel capacity, but the uncontrolled flow from Duck Creek through the Duck Creek Diversion Channel was more than the lower creek channels could carry. Flood damage was primarily concentrated about South Littlejohn Creek. On the south branch of the creek, the flood damaged barley crops, farm buildings, supplies and equipment. Flood damages on the north branch were primarily to residences and to small business establishments.

In the months preceding the April 1958 storm event, rainfall served to saturate the ground and increase the flood potential in the basin. Rainfall during January and February was about 200 percent of normal, totaling 11 inches (27.9 cm). During the two storm periods in March, there was an additional 6 inches (15.2 cm) of rain. For the period of 30 March through 6 April, a series of short and intense storms produced 6 inches (15.2 cm) of rain. The April floods were due to high flows and the inability of the local rainfall runoff to drain into the main channels. Sections of the natural sloughs and waterways were filled in, and the ground leveled for irrigation, without providing sufficient alternate drainage channels. The result was that about 2,000 acres of farmland were flooded. Depths of flooding varied from a few inches to two feet, with durations ranging from 12 hours to 10 days in ponded areas. Inundated crops included barley, alfalfa, and onions. There was also some damage to land from erosion, as well as to improvements and stored supplies. County roads also sustained fairly extensive damage.

In February 1986, the water level at Farmington Dam reached a high at elevation 155 feet. The flooded area behind the dam was completely drained within 13 days after this record flood event. For the period of 12-21 February, the Flowers Mountain precipitation gage received a total of 7.6 inches. The Stockton WSO Airport precipitation gage received a total of 5.98 inches, while a total of 5.88 inches was recorded for the Knights Ferry 2 ESE gage.

In February 1998, a succession of intense El Niño-driven storms swept over northern and central California for nearly four weeks. These cold storms, originating from the Gulf of Alaska, were accompanied by strong winds. The storms produced low snow levels and widespread showers and thunderstorms. In many areas the ground became nearly saturated due to the cumulative effect of the rains. According to NOAA, California experienced the wettest February on record. The Stockton WSO Airport precipitation gage received a total of 8.01 inches, approximately 360 percent of average. The Flowers Mountain precipitation gage received a rainfall amount totaling about 12.2 inches, approximately 330 percent of average. The Farmington Reservoir pool elevation reached 156.89 feet. This was the first time the pool elevation had exceeded the gross pool level since completion of the project. Farmington Dam and Reservoir were able to prevent an estimated $3.5 million in flood damages.

Table 10. Dams and Lakes in the San Joaquin River Basin

Dams and Lakes in the San Joaquin River Basin			
		Storage (Ac-Ft)	
Dam/Lake	Tributary Stream	Gross Pool	Owner / Operator
SAN JOAQUIN RIVER BASIN			
Camanche	Mokelumne River	417,000	EBMUD
New Hogan	Calaveras River	317,100	USACE
Farmington	Little John Creek	52,000	USACE
New Melones	Stanislaus River	2,420,000	USBR
Tulloch	Stanislaus River	67,000	USBR
Don Pedro	Tuolumne River	2,030,000	TID
New Exchequer/ McClure	Merced River	1,024,000	MID
Burns	Bear Creek / Merced Stream Group	6,800	USACE
Bear	Bear Creek / Merced Stream Group	7,700	USACE
Owens	Owens Creek / Merced Stream Group	3,600	USACE
Mariposa	Bear Creek / Merced Stream Group	15,000	USACE
Los Banos	Los Banos Creek	34,600	CA-DWR
Buchanan/Eastman	Chowcilla River	150,000	USACE
Hidden/Hensley	Fresno River	90,000	USACE
Friant/Millerton	San Joaquin River	520,500	USBR
Big Dry Creek	Big Dry Creek, tributary to the San Joaquin River	30,200	FMFCD
TULARE LAKEBED BASIN			
Pine Flat	Kings River	1,000,000	USACE
TOTAL SYSTEM STORAGE		8,185,500	

Key:

CA-DWR	California Department of Water Resources
EBMUD	East Bay Municipal Utilities District
FMFCD	Fresno Metropolitan Flood Control District
MID	Merced Irrigation District
TID	Turlock Irrigation District
USACE	US Army Corps of Engineers
USBR	US Bureau of Rclamation

Table 11. Historical Flooding on the Calaveras River

Historical Flooding in the Calaveras River (1 of 2)			
	Peak Flow (a) c.f.s.		
Flood	Recorded Peak Flow at Mormon Slough at Bellota	Natural Flow at Jenny Lind	Calaveras River at Jenny Lind
March 1907	(b)		34,600
January 1909	(b)		33,000
Jan-Feb 1911	(b)		50,000
January 1916	(b)		22,000
February 1917	(b)		31,300
March 1918	(b)		21,800
January 1921	(b)		37,900
February 1922	(b)		24,500
February 1925	(b)		27,500
February 1936	(b)	(37,000)	10,100
February 1938	(b)	(42,000)	10,600
Nov-Dec 1950	(9000)	(23,000)	7,600
December 1955	(16,000)	(33,000)	14,200
April 1958	15,400	(43,000)	12,100
February 1963	6,700	(25,000)	6,900
Dec 1964-Jan 1965	3,300	(33,000)	2,600
January 1969	10,700	(20,000)	(c)

Note: Neither the Jenny Lind gage nor the Bellota gage were in operation from February 1969 through March 1988.

Table 11. Historical Flooding on the Calaveras River

Historical Flooding in the Calaveras River (2 of 2)			
	Recorded Peak Flow at Mormon Slough at Bellota	**Natural Flow at Bellota**	**Date of Peak at Bellota**
April 1988	8,500	(8600)	22-Apr-88
June 1989	1,000	(900)	9-Jun-89
August 1990	1,200	(1200)	3-Mar-90
May 1991	7,900	(7900)	14-May-91
June 1992	4,100	(7000)	15-Feb-92
May 1993	7,600	(7600)	5-May-93
October 1993	1,800	Missing	(d)
May 1996	3,000	(10200)	21-Feb-96
January 1997	7,800	(29600)	2-Jan-97
February 1998	9,600	(40800)	3-Feb-98
February 1999	6,800	(19900)	9-Feb-99
February 2000	4,500	(16000)	25-Jan-00
March 2001	2,200	(5500)	5-Mar-01
January 2002	2,100	(6200)	3-Jan-02
December 2002	700	(4700)	16-Dec-02
February 2004	3,500	(6700)	2-Jan-04
March 2005	4,400	(14500)	23-Mar-05
April 2006	9,500	(32600)	4-Apr-06
February 2007	1,400	(6100)	27-Feb-07
January 2008	1,300	(5700)	28-Jan-08
March 2009	1,000	(10300)	4-Mar-09
January 2010	2,300	(6600)	22-Jan-10
March 2011	8,900	(18200)	20-Mar-11
April 2012	1,700	(6800)	13-Apr-12

(a) Flow values shown in ( ) are estimated. For the Jenny Lind station (1969 and prior), estimated peaks remove the effect of old Hogan dam (1936-1963) or New Hogan dam (1964-present); recorded flows are also shown for comparison. All flows are rounded.
(b) Station not in operation.
(c) Station discontinued.
(d) Station operated by USACE 1988 to 1996 with daily values and from 1996 to present with hourly values. Daily and hourly values from 1998 to present are observed flows affected by regulation of New Hogan dam. Natural peak flows () at Bellota are estimated from 1988 to 1995.
Source: New Hogan Water Control Manual, June 1983, and USACE DSS files.

Table 12. Drainage Area at Selected Locations in the San Joaquin River Basin

Drainage Area of Selected Locations in the San Joaquin River Basin
and Drainage Area Controlled by Upstream Dams
in upstream to downstream order

SAN JOAQUIN RIVER BASIN

USGS Station No.	Location / Dam and Lake	Tributary Stream	Drainage Area	Percent of dA Controlled
11221500	**Pine Flat Lake & Dam**	Kings River	**1545**	100%
11222000	at Piedra	Kings River	1693	91%
11250999	**Friant Dam/Millerton Lake**	San Joaquin River	**1638**	100%
11254001	at Mendota	San Joaquin River	3943	81%
11257999	**Hidden/Hensley**	Fresno River	**236**	100%
11258000	below Hidden dam near Daulton gage	Fresno River	258	91%
11258001	at East Side Bypass (approx)	Fresno River	480	49%
11258999	**Buchanan/Eastman**	Chowcilla River	**235**	100%
11259999	at East Side Bypass (approx)	Chowcilla River	600	39%
11260000	'at El Nido	San Joaquin River	6443	57%
11260288	**Burns**	Bear Creek / Merced Stream Group	**71.9**	100%
11260289	**Bear**	Bear Creek / Merced Stream Group	**72.3**	100%
11260291	**Owens**	Owens Creek / Merced Stream Group	**25.7**	100%
11260292	**Mariposa**	Bear Creek / Merced Stream Group	**108.5**	100%
11261500	at Fremont Ford Bridge	San Joaquin River	7615	52%
11262799	**Los Banos damsite**	Los Banos Creek	**156**	100%
11262800	near Los Banos	Los Banos Creek	159	98%
11273400	above Merced River near Newman	San Joaquin River	7949	51%
11270000	**New Exchequer/ McClure**	Merced River	**1037**	100%
11270610	at McSwain Dam	Merced River	1054	98%
11272500	at Stevinson	Merced River	1273	81%
11273500	at mouth of Merced at River Road Bridge	Merced River	1276	81%
11274000	near Newman	San Joaquin River	9520	54%
11274550	near Crows Landing	San Joaquin River	9694	53%
11274570	at Patterson Bridge near Patterson	San Joaquin River	9749	53%
11288000	**Don Pedro abv LaGrange Dam**	Tuolumne River	**1533**	100%
11290000	at Modesto	Tuolumne River	1884	81%
11290200	at Shiloh Road Bridge nr Grayson	Tuolumne River	1897	81%
11299200	**New Melones**	Stanislaus River	**904**	100%
11302000	below Goodwin Dam near Knights Ferry	Stanislaus River	986	92%
11302500	at Oakdale	Stanislaus River	1032	88%
11303000	at Ripon	Stanislaus River	1075	84%
11303500	***at Vernalis***	***San Joaquin River***	***13536***	***56%***
11308900	**New Hogan**	Calaveras River	**363**	100%
11309500	at Jenny Lind	Calaveras River	393	92%
11309599	Mormon Slough at Bellota	Calaveras River	470	77%
11309601	**Farmington**	Little John Creek	**212**	100%
11309602	at Farmington	Little John Creek	247.9	86%
11323500	**Camanche**	Mokelumne River	**621**	100%
11325500	at Woodbridge	Mokelumne River	661	94%

9.0 DELTA BASE FLOOD ELEVATION, TIDE STAGE FREQUENCY ANALYSIS

A stage frequency analysis was needed for Delta near Stockton. Initially, the analysis was described briefly in the hydrology appendix and focused on two key delta stage gages near Stockton called Rindge Pump gage and Burns Cutoff gage as shown in Figure 9.1. Recently, the Delta stage frequency analysis was moved to the Hydraulics Appendix. Please refer to the Hydraulics Appendix for further details of that analysis.

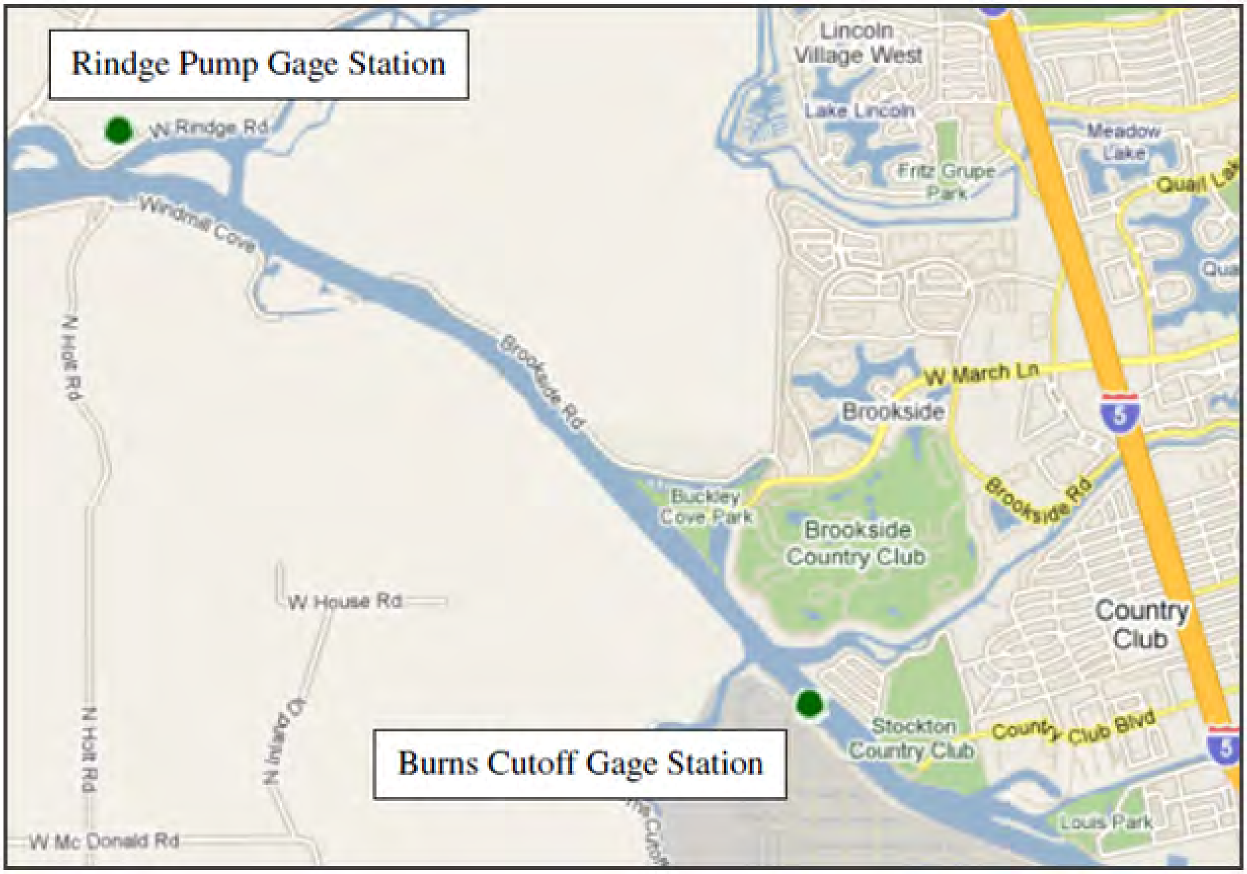

Figure 9.1 – Rindge Pump and Burns Cutoff Gage Station Location Map

10.0 HYDROLOGIC ANALYSIS OF ALTERNATIVES

None of the alternatives presently under consideration will have an effect on the existing or future condition hydrology of the basins and/or river reaches within the study area.

The operation of New Hogan dam was analyzed to determine the level of protection of the dam. The flow-frequency analysis shows that there is a 0.5 (1/200) ACE level of protection in the current operation of the dam and that no changes in operation are required to achieve the state goal of 1/200 year level of protection. The 1958 flood event was the only event in history that produced a spillway event. The New Hogan dam was not constructed until 1963, so the original (smaller) Hogan dam allowed that spillway event and consequential flooding. It was found that the flood control storage capacity of the reservoir lies between the 0.5 (1/200) ACE 3-day inflow volume and the 0.5 (1/200) ACE 4-day inflow volume. However, none of the historic events exceeded to total required storage volume. Therefore, a dam raise was considered infeasible. This analysis was done from a hydrologic perspective only and does not constitute a thorough reservoir re-operation or dam safety investigation as required by regulations. The details of the analysis are further described in a technical memorandum prepared for the LSJR feasibility study by David Ford Consulting Engineers in August of 2011 (Ford, 2011).

The State of California through the FloodSAFE program and the Central Valley Flood Protection Plan (CVFPP) will be studying the potential for re-operation of the flood control projects throughout the central valley. Because the Corps of Engineers has section 7 of the flood control act of 1944 authority over flood control operations, the Corps will engage with the state at an appropriate time. That analysis is not part of this feasibility study and the results will not be known for several years. Further information is available on the DWR website at: http://www.water.ca.gov/system_reop/.

The U.S. Bureau of Reclamation has underway a feasibility study for a new dam upstream of Friant dam and Millerton Lake on the upper San Joaquin river. The Temperance Flat project will provide additional flood protection to the study area, however, construction of the dam is in the future and cannot be considered in the future without project condition of this study. Further information is available online at: http://www.usbr.gov/mp/sccao/storage/docs/phase1_rpt_fnl/.

The U.S. Fish and Wildlife Service is performing a conservation study looking at alternatives for habitat and ecosystem restoration in the upper and lower San Joaquin River corridor. That study may provide additional flood protection benefits to the study area. However, those projects also cannot be considered part of the future without project condition. Further information is available at: http://www.fws.gov/sacramento/Fisheries/San-Joaquin/fisheries_san-joaquin.htm.

11.0 RESULTS AND CONCLUSIONS

A description of the study area, flood history and flood problems, and flood control projects has been presented.

The results of the design storm analysis, the unregulated flow frequency of Bear Creek at Lockeford, Cosgrove Creek at Valley Springs, the Calaveras River at New Hogan and Bellota, and Littlejohn Creek at Farmington Dam and at Farmington, and the San Joaquin River at Vernalis are provided.

In addition existing and future condition without project flows are provided at the damage index points that are shared with the hydraulic analysis, geotechnical analysis, and economic analysis teams.

The following technical memorandums are attached by reference as appendices to this summary hydrologic report:

1) Calaveras River watershed above Bellota hydrologic analysis, by USACE dated April, 2014.
2) Littlejohn Creek above Farmington, Ca hydrologic analysis by USACE, April 2014.
3) USACE Addendum to PBI Report, dated April 2014.
4) The Sacramento – San Joaquin Comprehensive Study, Technical Studies Documentation: Appendices A through D, USACE, 2002.
   On the world-wide-web at: http://130.165.3.37/reports.html
5) New Hogan Dam Water Control Manual, USACE, 1983.
6) Farming Dam Water Control Manual, USACE, 2004.

12.0 REFERENCES:

[1] United States Army Corps of Engineers (USACE), 2002, Sacramento – San Joaquin River Basins Comprehensive Study (Comp Study), California, Interim Report and Appendices.

[2] USACE, 2008, Lower San Joaquin River, California Feasibility Study, Project Management Plan.

[3] USACE, 2004, Lower San Joaquin River, California Section 905(b) Analysis (WRDA 1986).

[4] Peterson-Brustad, Inc (PBI), 2012, Lower San Joaquin River Feasibility Study, F3 Hydrology Appendix.

[5] Peterson-Brustad, Inc (PBI), 2010, San Joaquin River delta Base Flood Elevation Refinement Stage Frequency Analysis.

[6] David Ford Consulting Engineers Inc (Ford), 2011, Lower San Joaquin River Feasibility Study, Calaveras River frequency analysis and hydrographs.

[7] David Ford Consulting Engineers Inc (Ford), 2011, Lower San Joaquin River Feasibility Study, Littlejohn Creek frequency analysis and hydrographs.

[8] National Oceanic and Atmospheric Administration (NOAA), 2011, NOAA Atlas 14, Precipitation-Frequency Atlas of the United States, Volume 6 version 2.0 California.

[9] USACE, 2009, Guadalupe Watershed Hydrologic Assessment, Final Report.

[10] USACE, 1999, Sacramento and San Joaquin River Basins, California, Post - Flood Assessment.

[11] USACE, 1983, New Hogan Dam and Lake, Calaveras River, California, Water Control Manual.

[12] USACE, 2004, Farmington Dam and Reservoir, Littlejohn Creek, California, Water Control Manual.

[13] San Joaquin Area Flood Control Agency (SJAFCA), 1998, Flood Protection Restoration Project, Final Technical Memorandums; No.1 Hydrology, No. 2 Hydraulics, No. 7 Residual Floodplains.

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SEE PLATES BELOW

P L A T E S

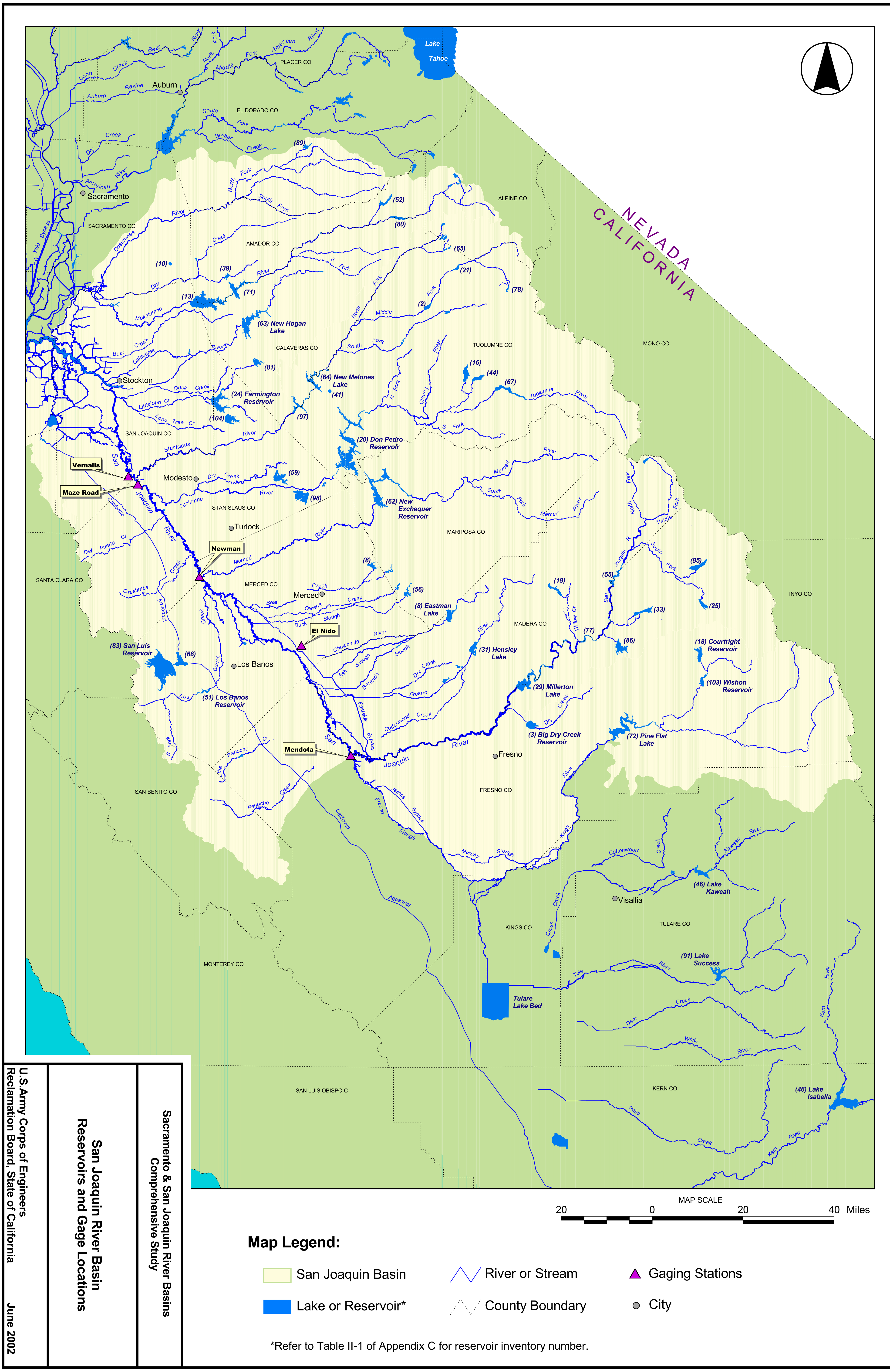


Plate 1. San Joaquin Basin Reservoir and Gage Location, from Comp Study

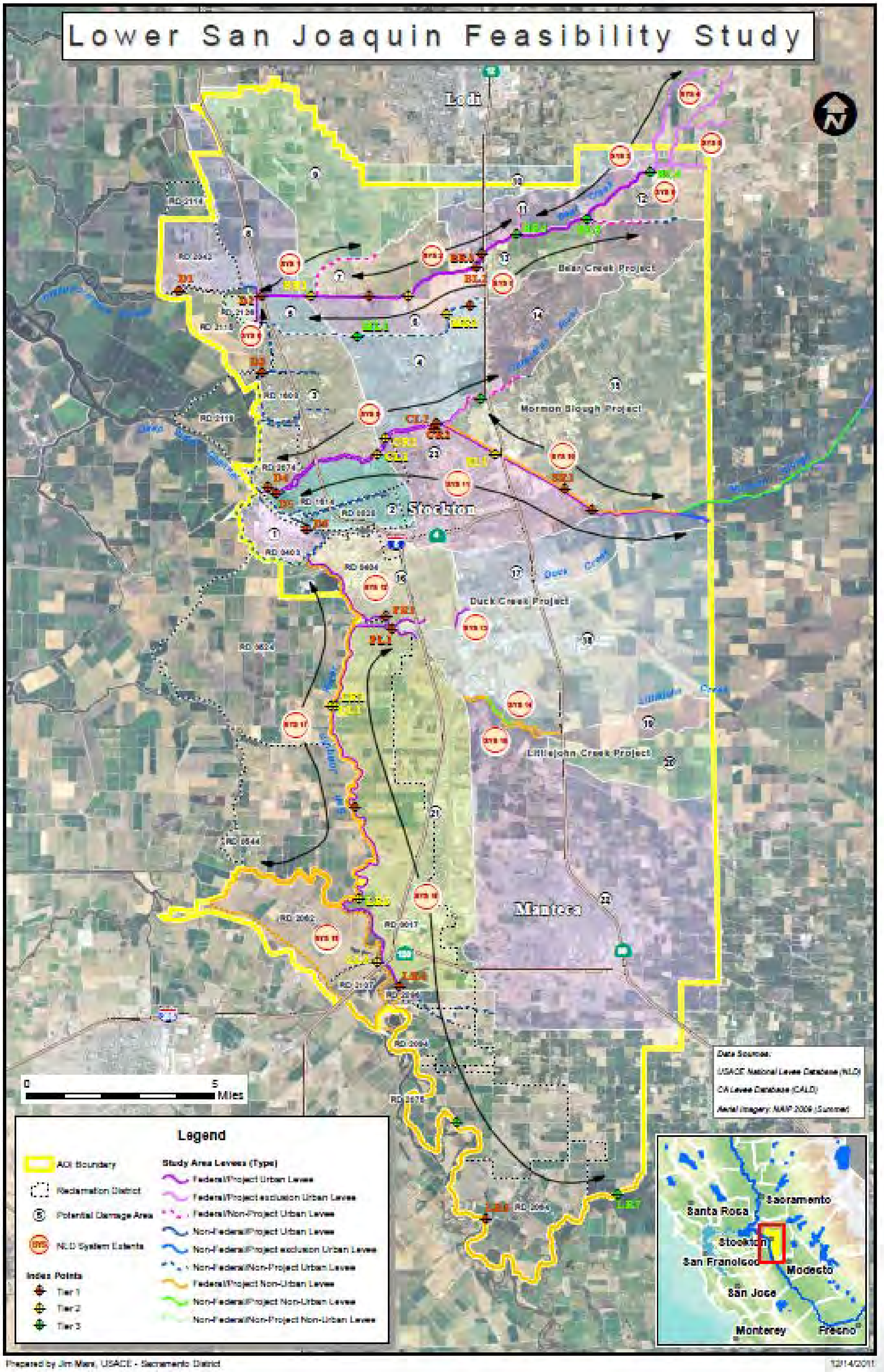


Plate 2. Lower San Joaquin Feasibility Study Area December 2011

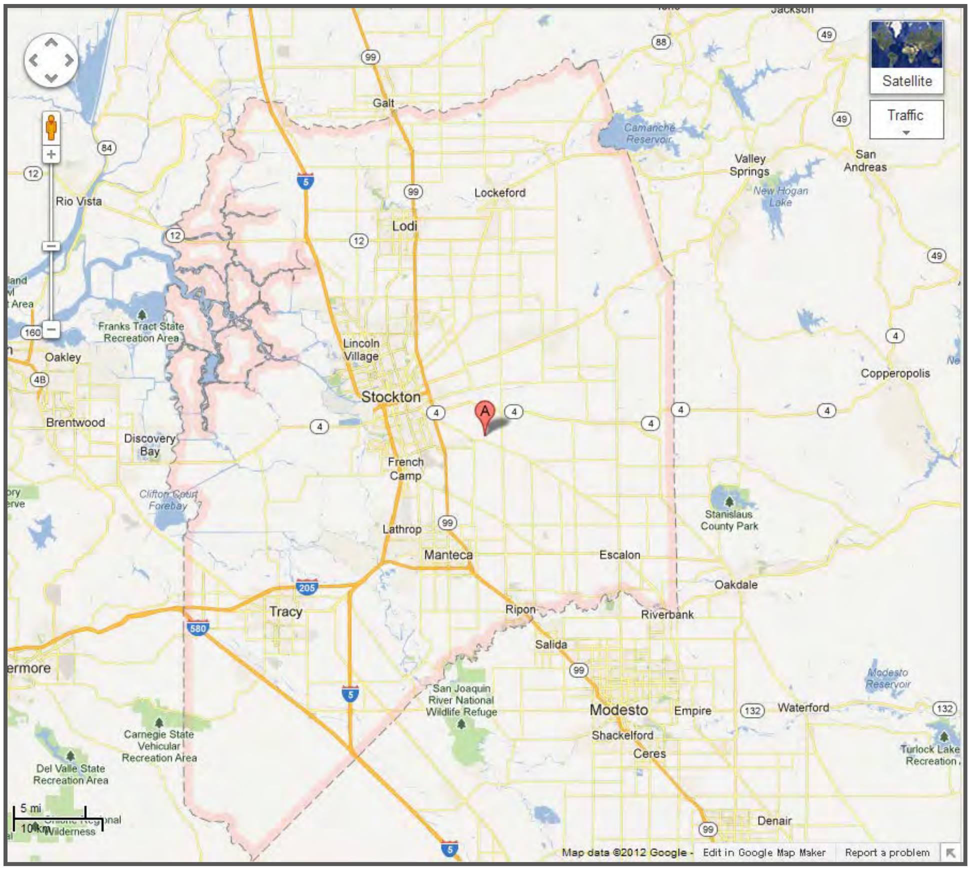


Plate 3. San Joaquin County, California boundary

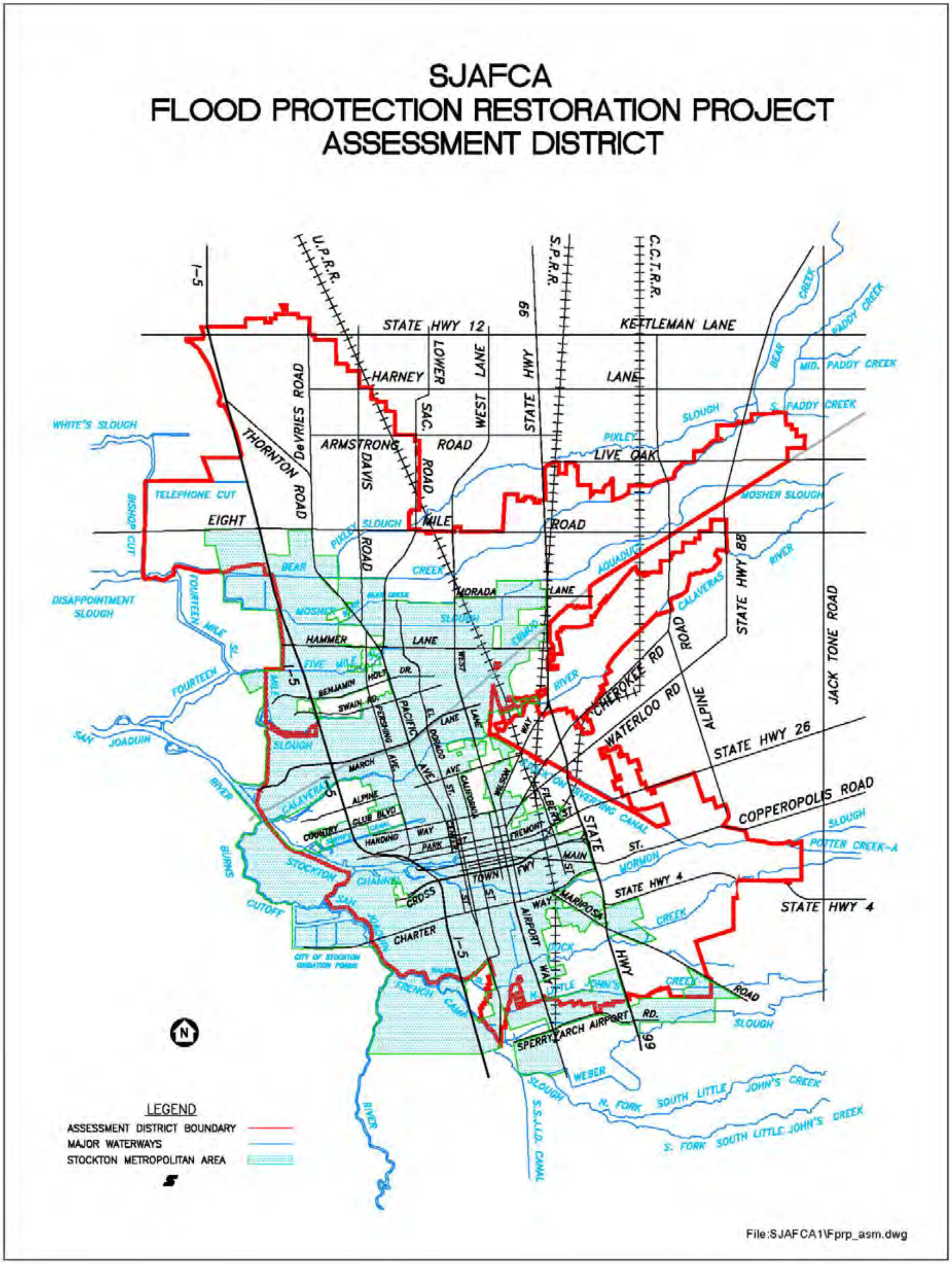


Plate 4. SJAFCA Boundary

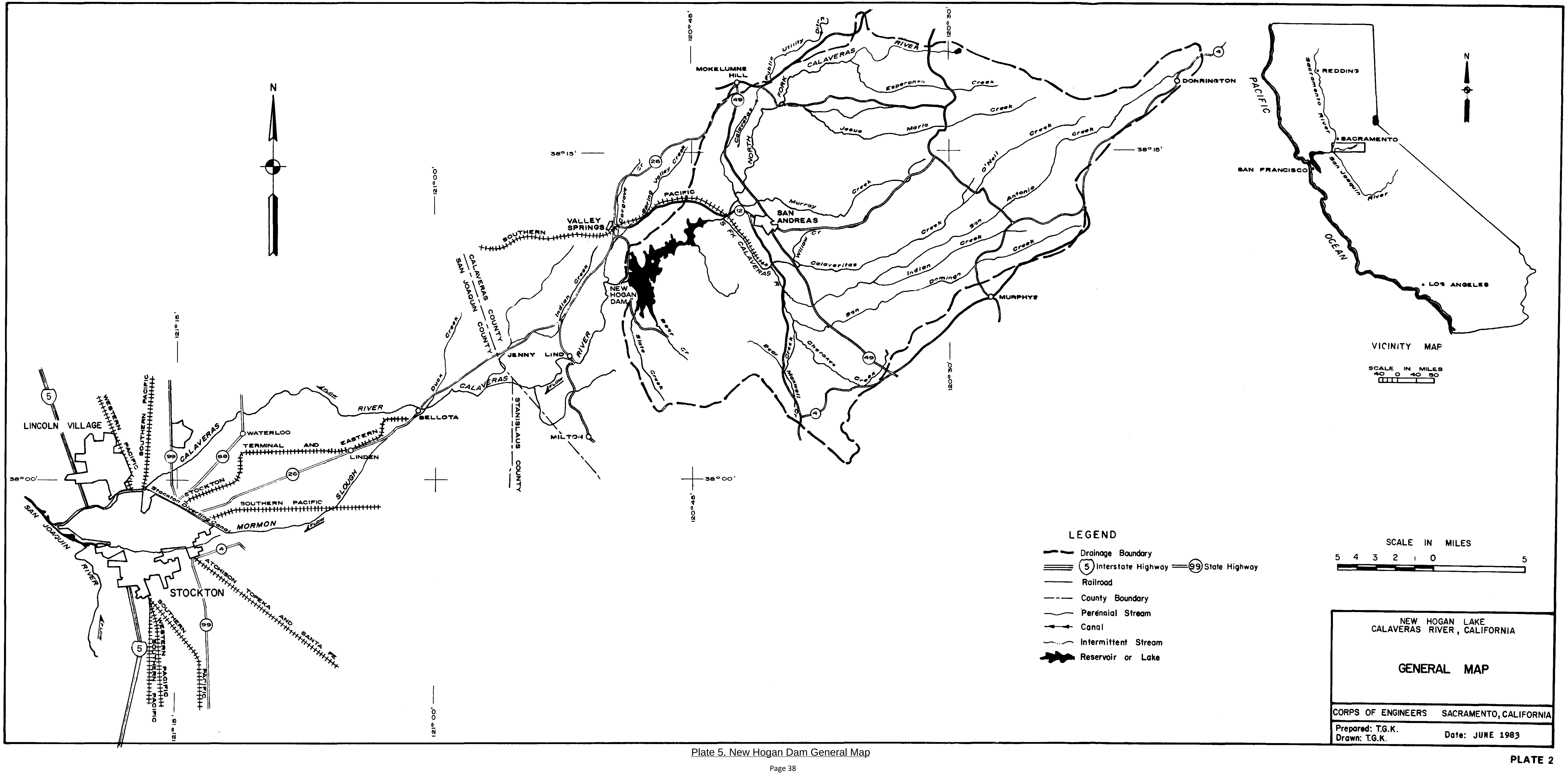


Plate 5. New Hogan Dam General Map

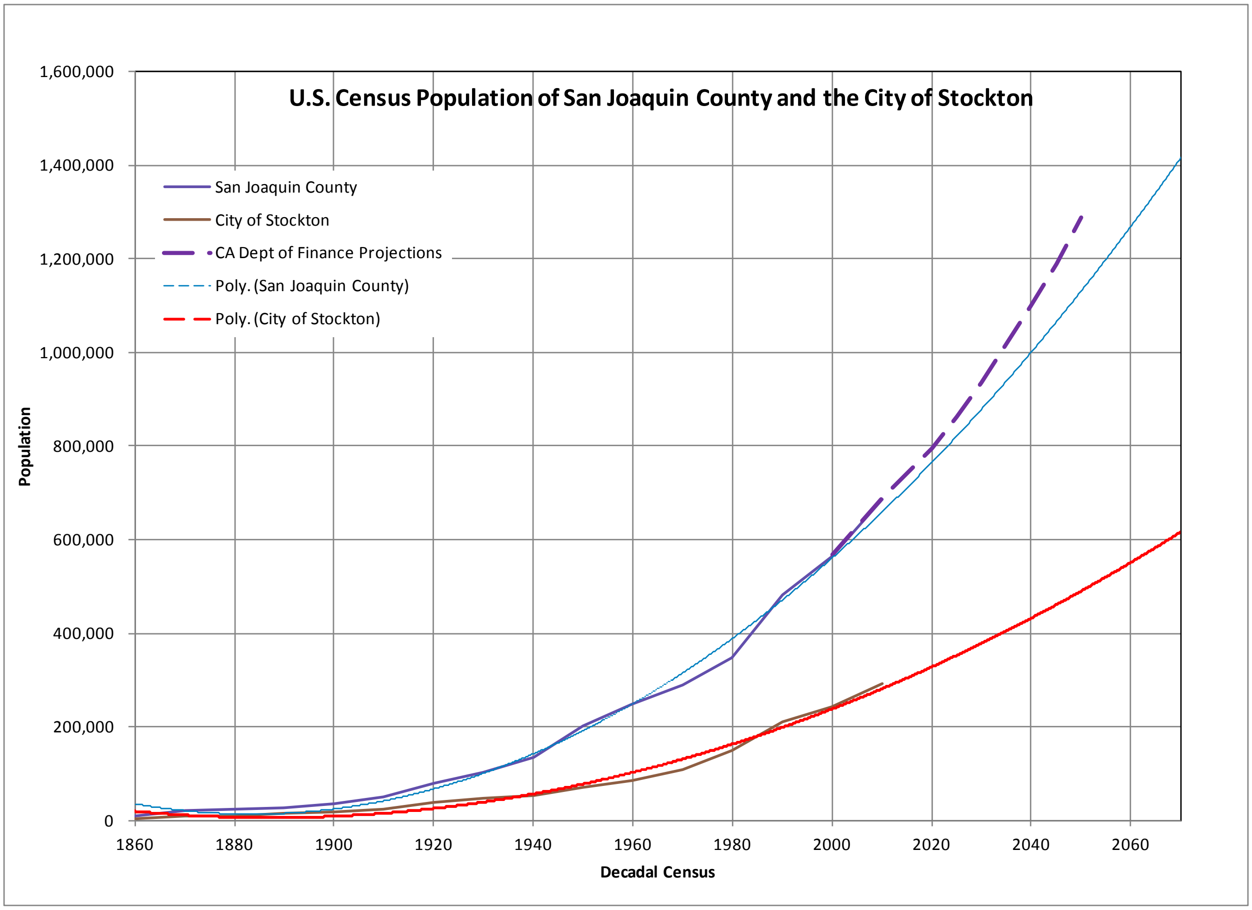
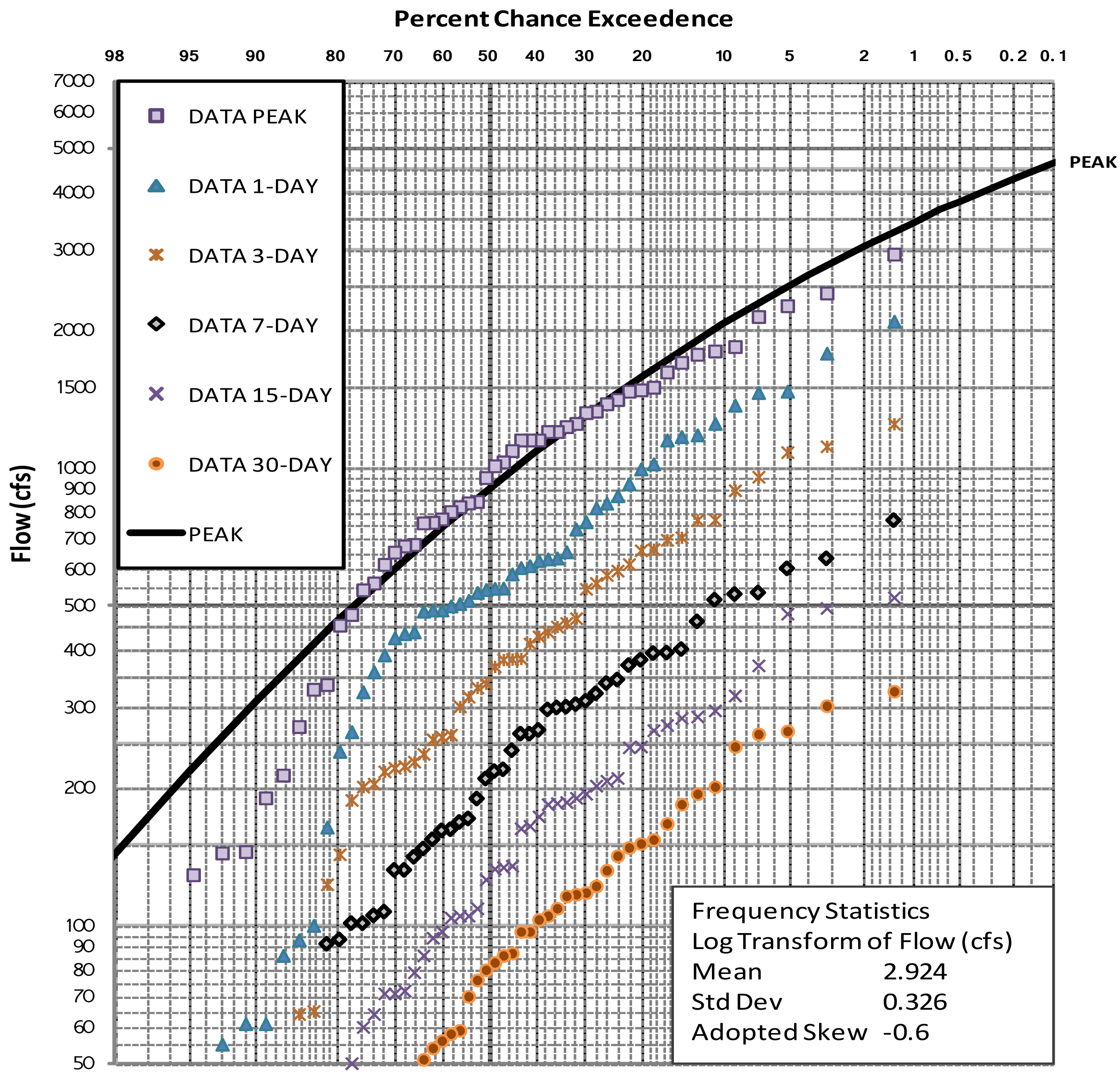


Plate 6. San Joaquin and Stockton Population 1960-2010 and Projection to 2070



NOTES:

1. Median plotting positions.
2. Computed Probability
3. Drainage area: 47.6 sq. mi.
4. WY 1932-1975, 1978-1985

LOWER SAN JOAQUIN FEASIBILITY STUDY
SAN JOAQUIN COUNTY, CALIFORNIA

BEAR CREEK AT LOCKEFORD
FLOW FREQUENCY CURVE

UNREGULATED (NATURAL) CONDITION

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

6-Dec-12

Plate 7. Analytical Flow Frequency at Bear Creek at Lockeford

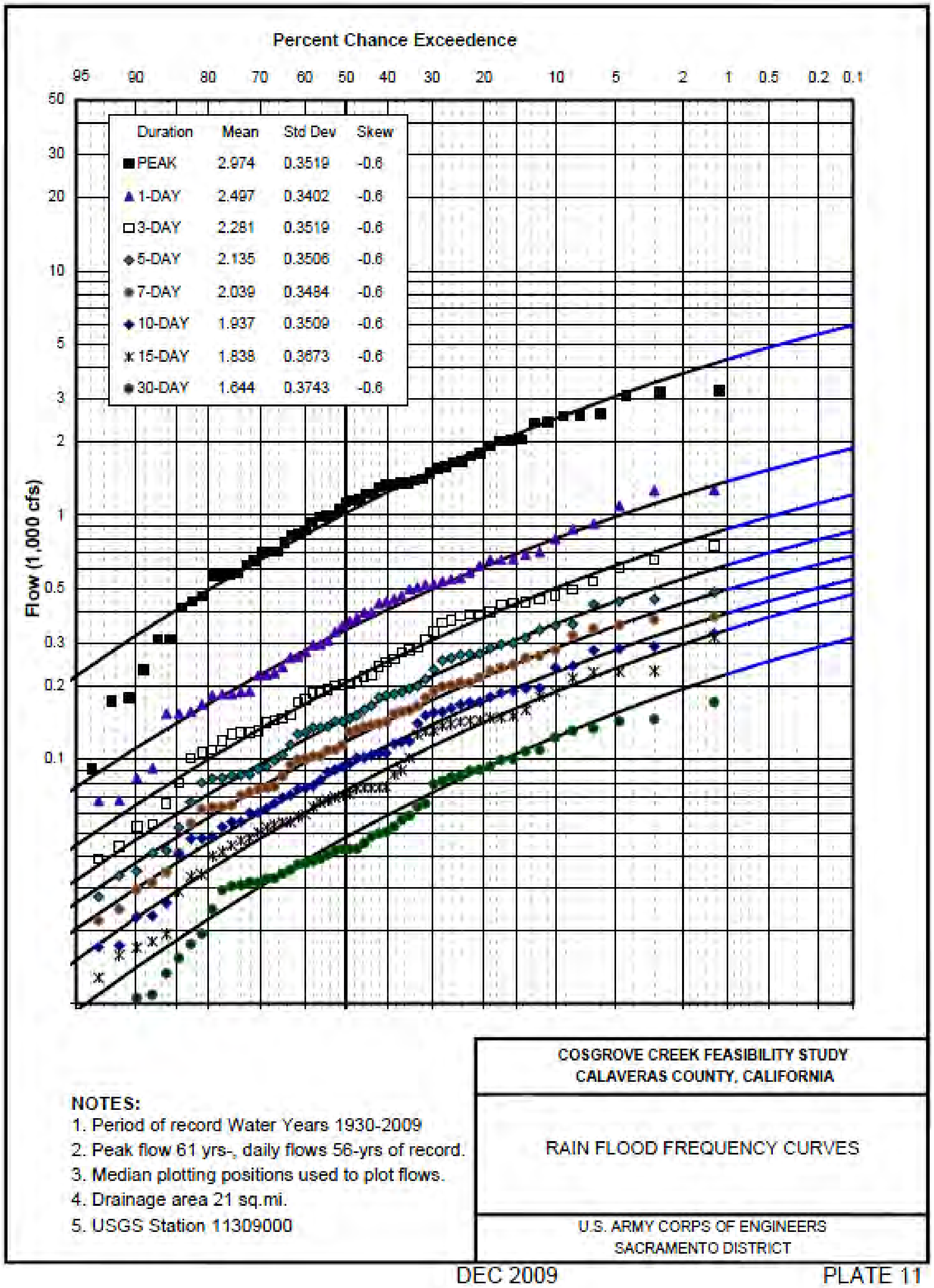
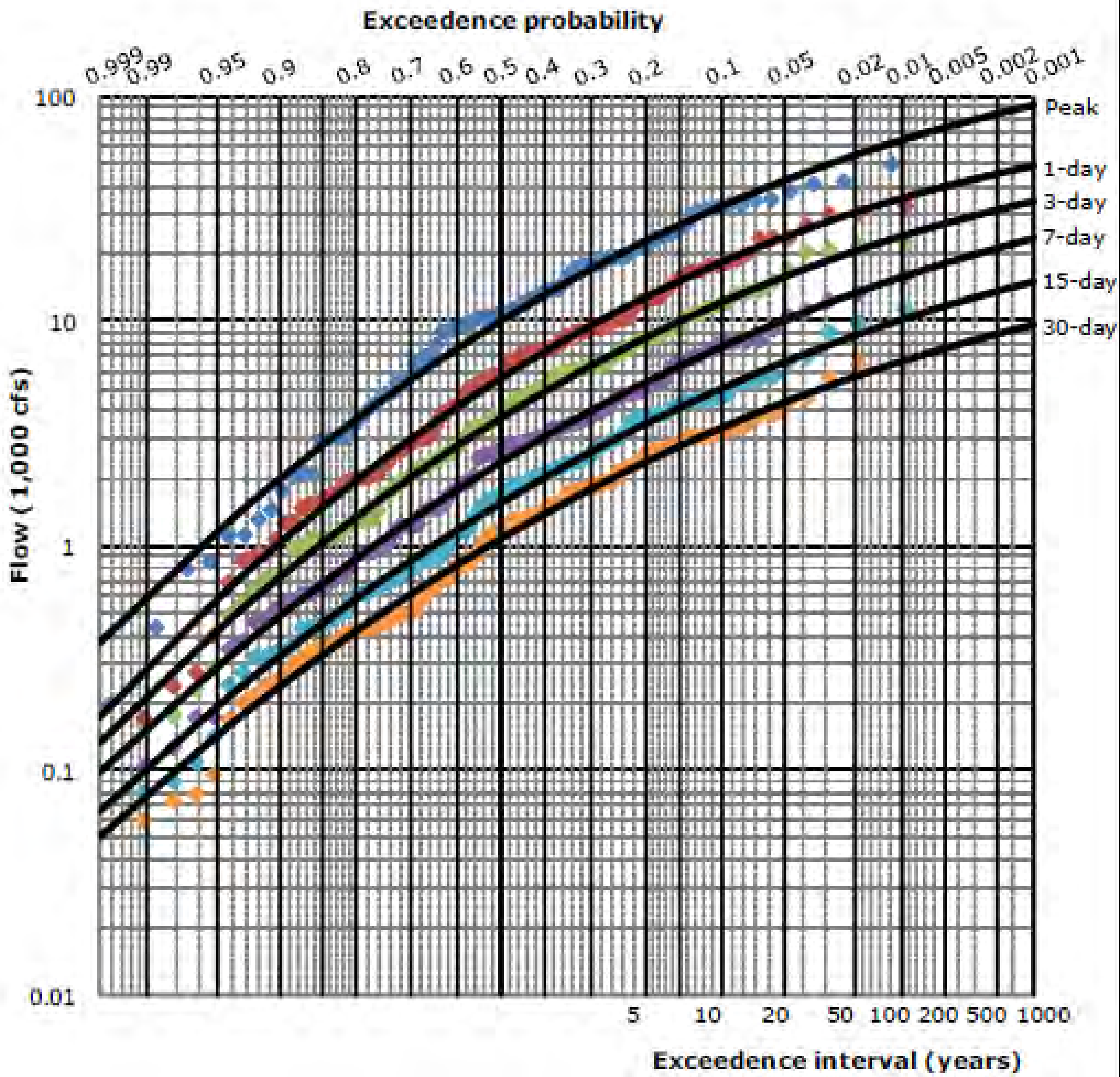


Plate 8. Analytical Flow Frequency at Cosgrove Creek at Valley Springs



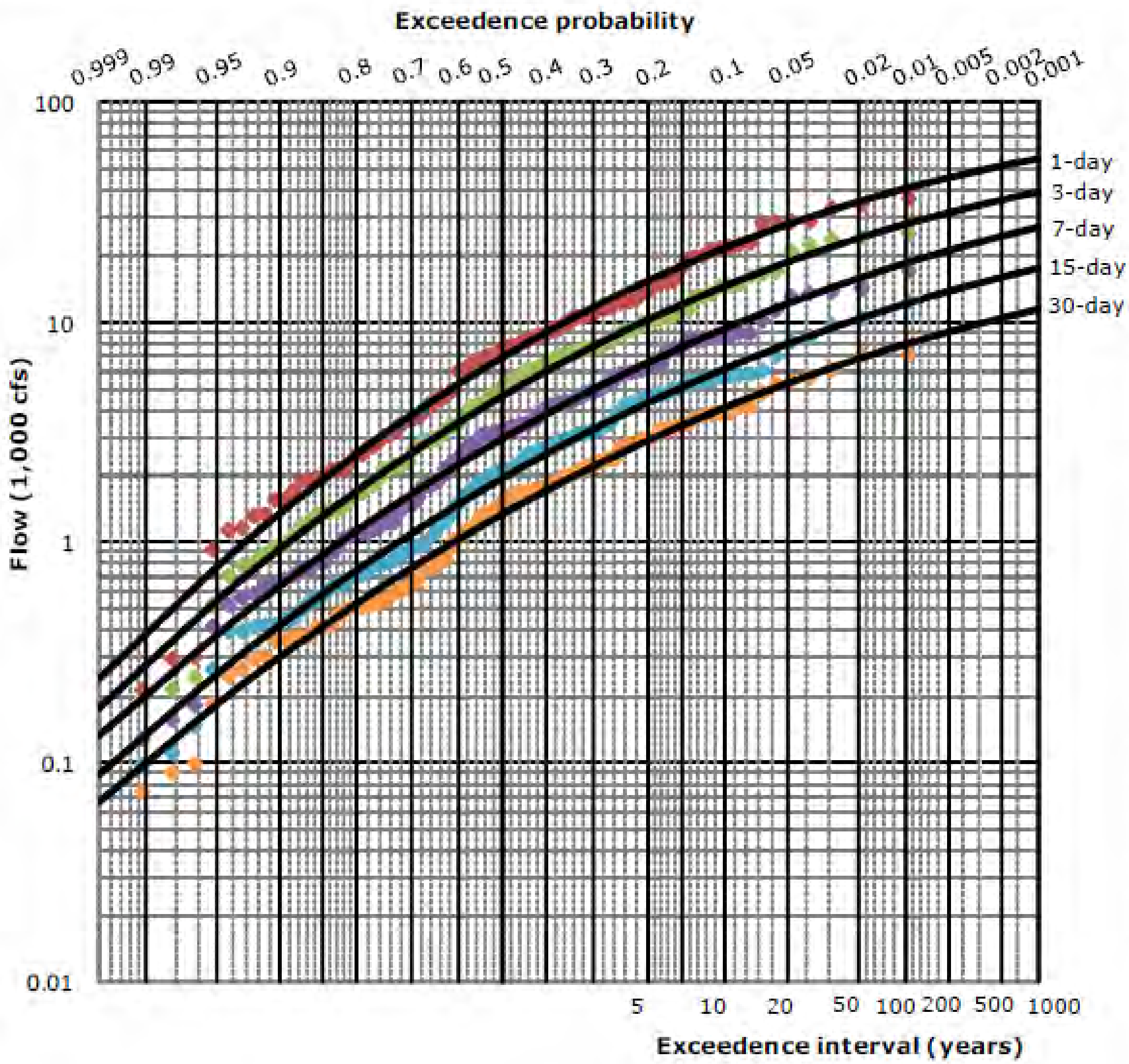
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.946	0.482	-0.727
1-day	3.685	0.501	-0.794
3-day	3.518	0.487	-0.732
7-day	3.324	0.477	-0.651
15-day	3.146	0.473	-0.656
30-day	2.988	0.457	-0.659

Notes:

- Median plotting positions.
- Drainage area: 363 sq. miles.
- Period of systematic record: 1907-2010.
  (Peak flow data intermittent 1930-2010).
- Record lengths
  Peak flows: 86 years.
  Volumes: 104 years.
- Regional skew values developed by USGS.

Plate 9. Analytical Unregulated Flow Frequency at New Hogan Dam



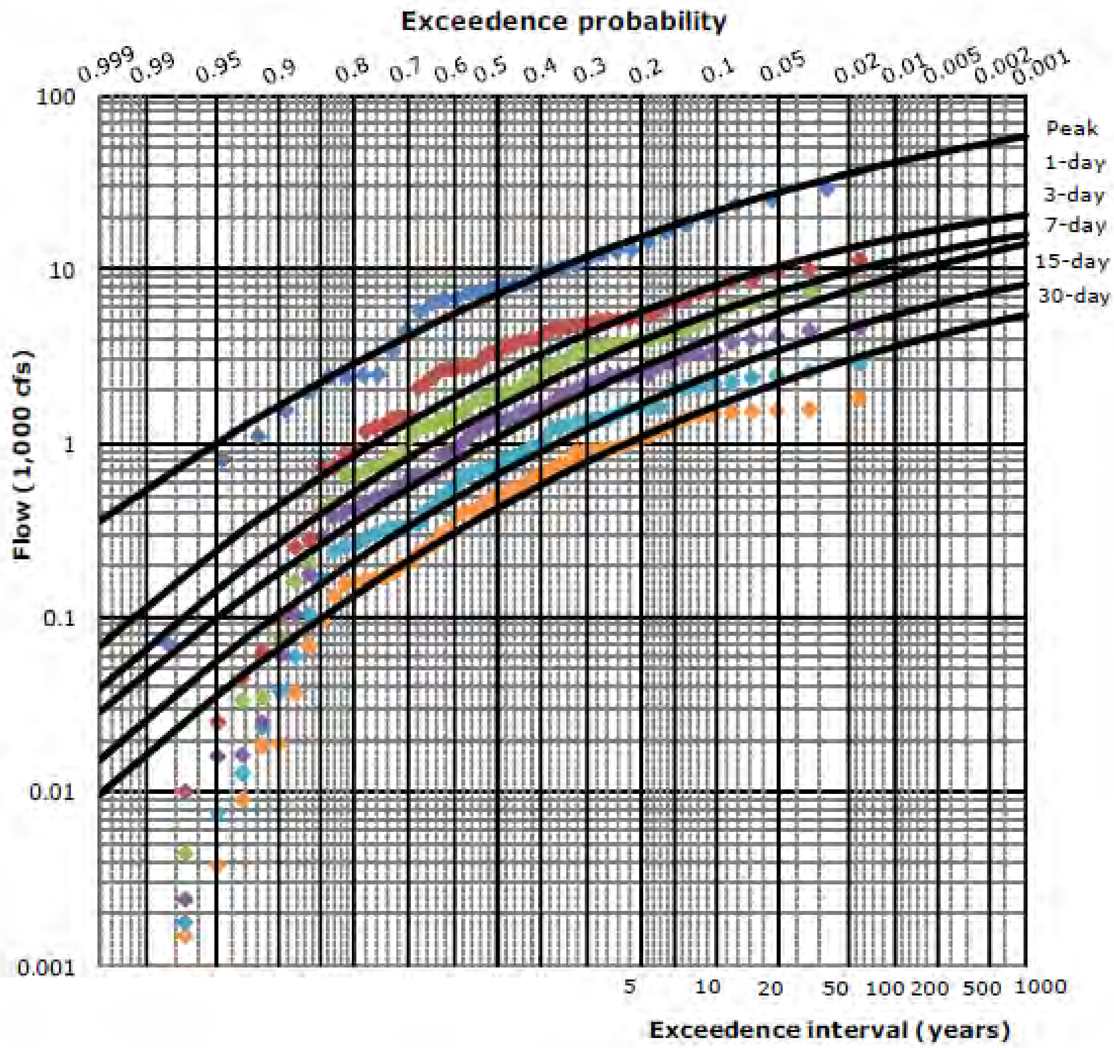
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
1-day	3.775	0.482	-0.810
3-day	3.608	0.475	-0.753
7-day	3.417	0.464	-0.666
15-day	3.240	0.461	-0.671
30-day	3.079	0.448	-0.668

Notes:

- Median plotting positions.
- Drainage area: 473 sq. miles.
- Period of systematic record: 1907-2010.
- Record length: 104 years.
- Regional skew values developed by USGS.

Plate 10. Analytical Unregulated Flow Frequency at Mormon Slough at Bellota



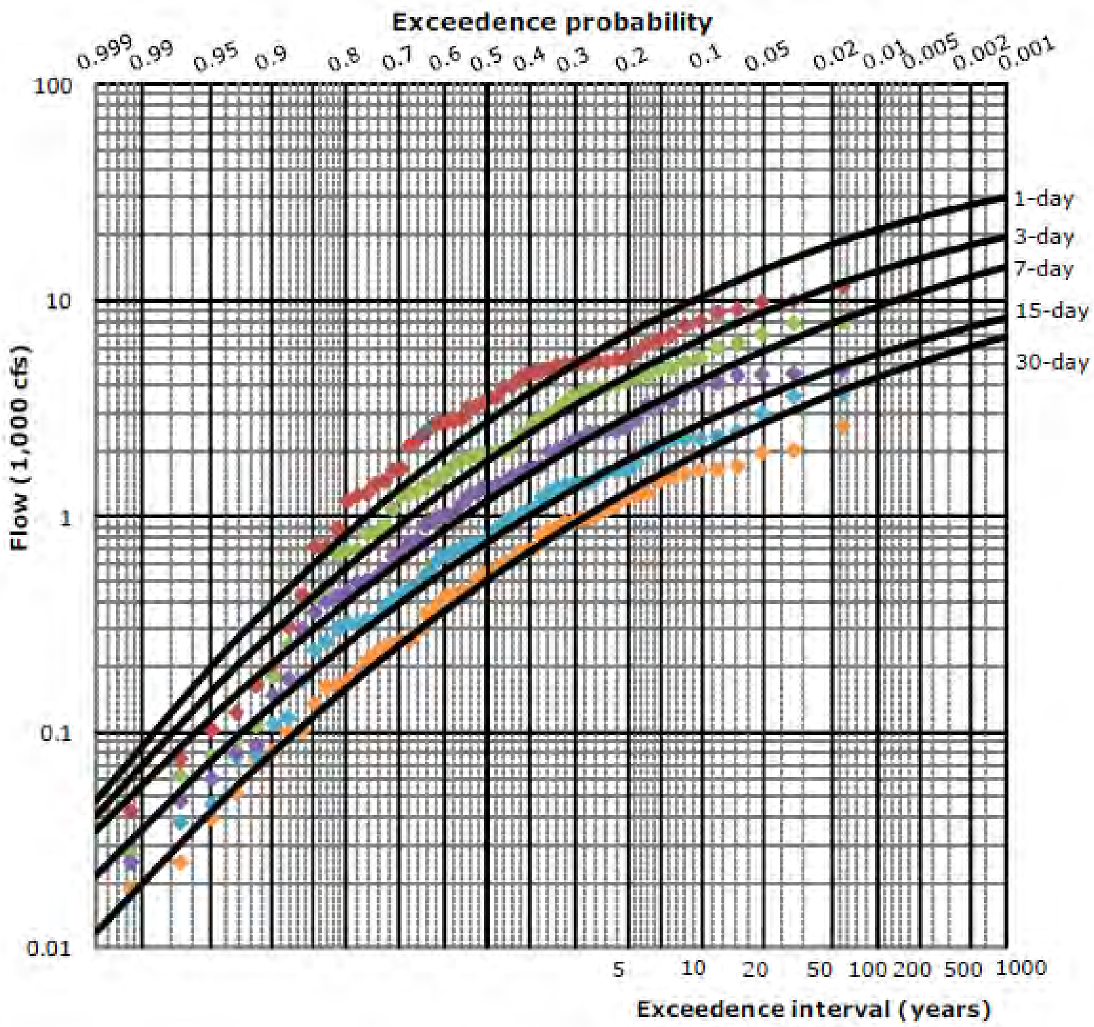
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.811	0.445	-0.692
1-day	3.321	0.507	-0.858
3-day	3.135	0.531	-0.812
7-day	2.970	0.538	-0.675
15-day	2.754	0.553	-0.733
30-day	2.561	0.556	-0.721

Notes:

- Median plotting positions.
- Drainage area: 212 sq. miles.
- Period of systematic record: 1951-2008.
  (Peak flow data intermittent 1952-2004).
- Record lengths
  Peak flows: 53 years.
  Volumes: 58 years.
- Regional skew values developed by USGS.
- Low outliers for volumes: 8 smallest events.

Plate 11. Analytical Unregulated Flow Frequency at Farmington Dam



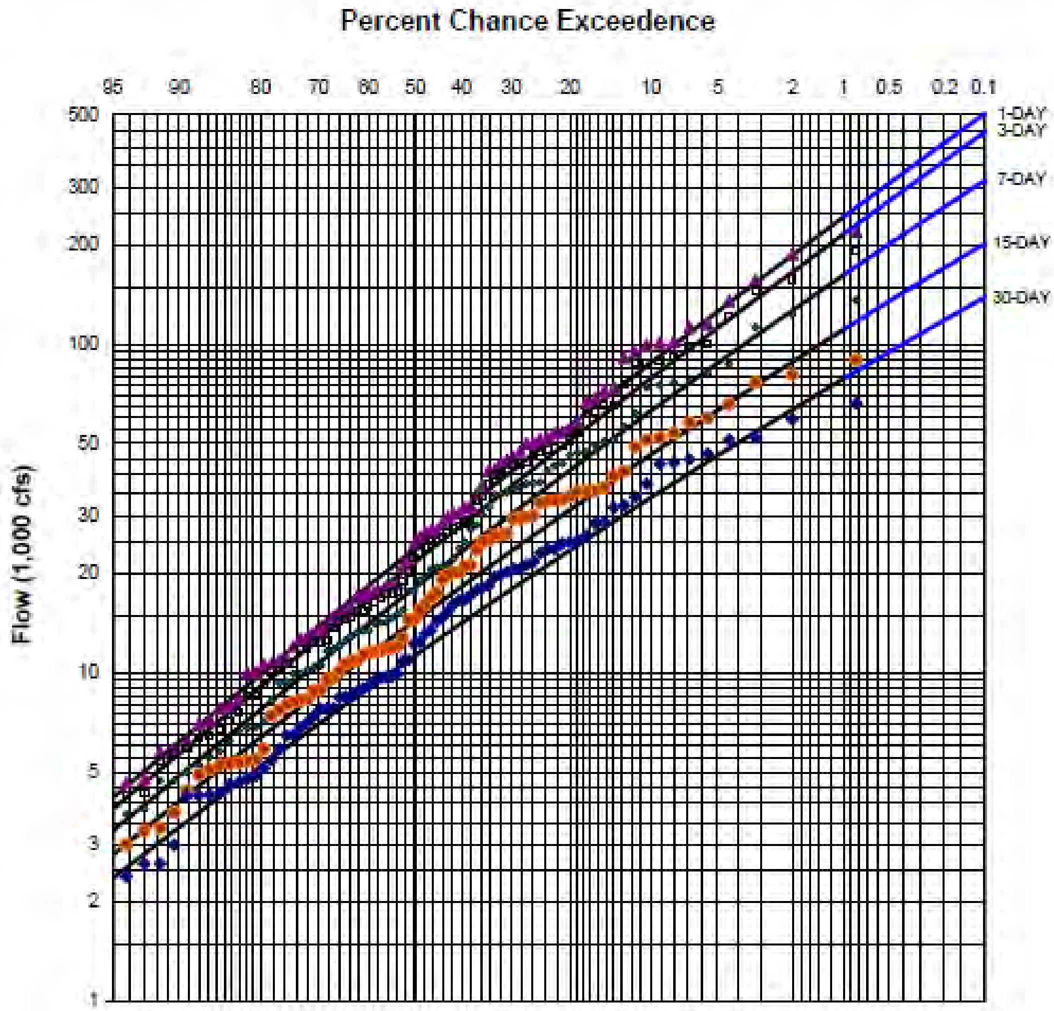
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
1-day	3.356	0.573	-0.849
3-day	3.186	0.545	-0.786
7-day	3.011	0.525	-0.670
15-day	2.815	0.523	-0.722
30-day	2.639	0.556	-0.695

Notes:

- Median plotting positions.
- Drainage area: 219 sq. miles.
- Period of systematic record: 1951-2008.
- Record length: 58 years.
- Regional skew values developed by USGS.
- Low outliers for 1-, 3, 7, and 15-day volumes: 7 smallest events.
- Low outliers for 30-day volumes: 6 smallest events.

Plate 12. Analytical Unregulated Flow Frequency at Littlejohn Creek at Farmington



ADOPTED STATISTICS:

	Mean	Std.Dev.	Skew
1-day	4.375	0.450	-0.1
3-day	4.333	0.445	-0.1
7-day	4.251	0.433	-0.2
15-day	4.148	0.412	-0.2
30-day	4.042	0.392	-0.2

NOTES:

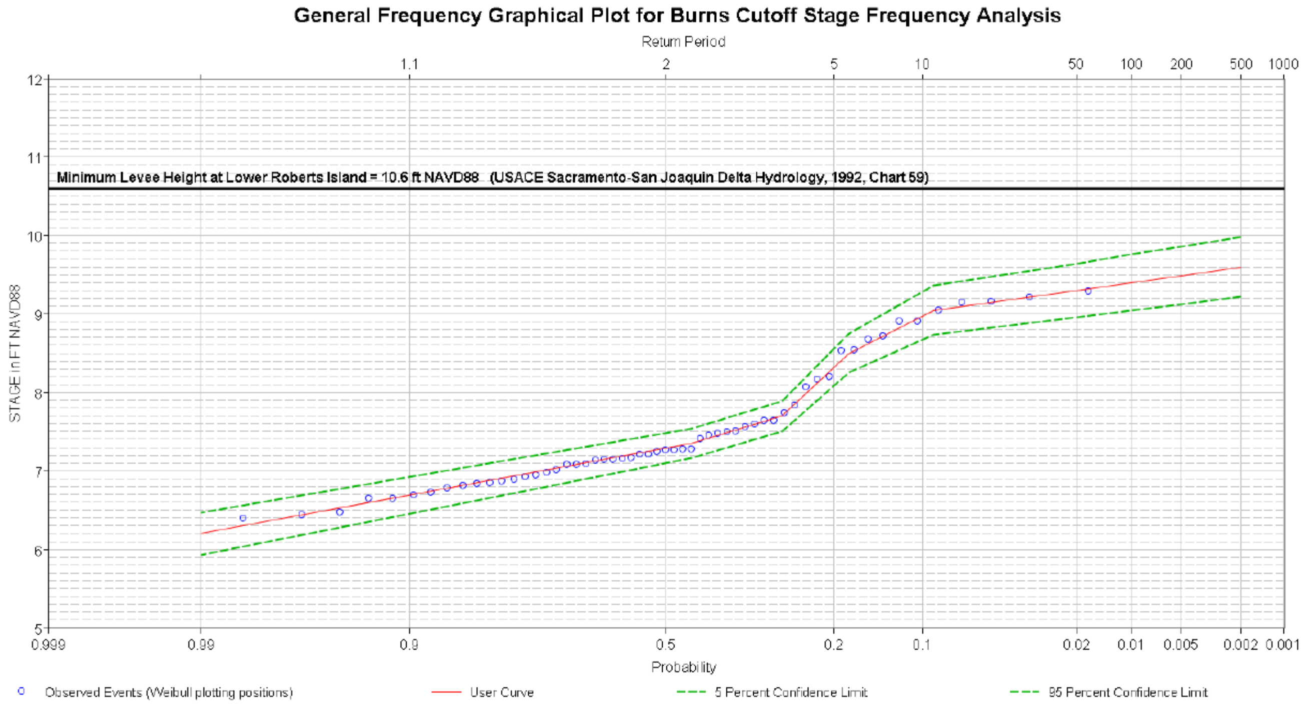
1. No overbank flow.
2. Median plotting positions.
3. Drainage area: 13,536 sq. mi.
4. Period of record: 1917-1998.

SACRAMENTO-SAN JOAQUIN COMPREHENSIVE STUDY
SAN JOAQUIN RIVER BASIN, CALIFORNIA

RAIN FLOOD FREQUENCY CURVES
SAN JOAQUIN RIVER NEAR VERNALIS
UNREGULATED CONDITIONS

U.S ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Plate 13. Analytical Unregulated Flow Frequency for the San Joaquin River at Vernalis



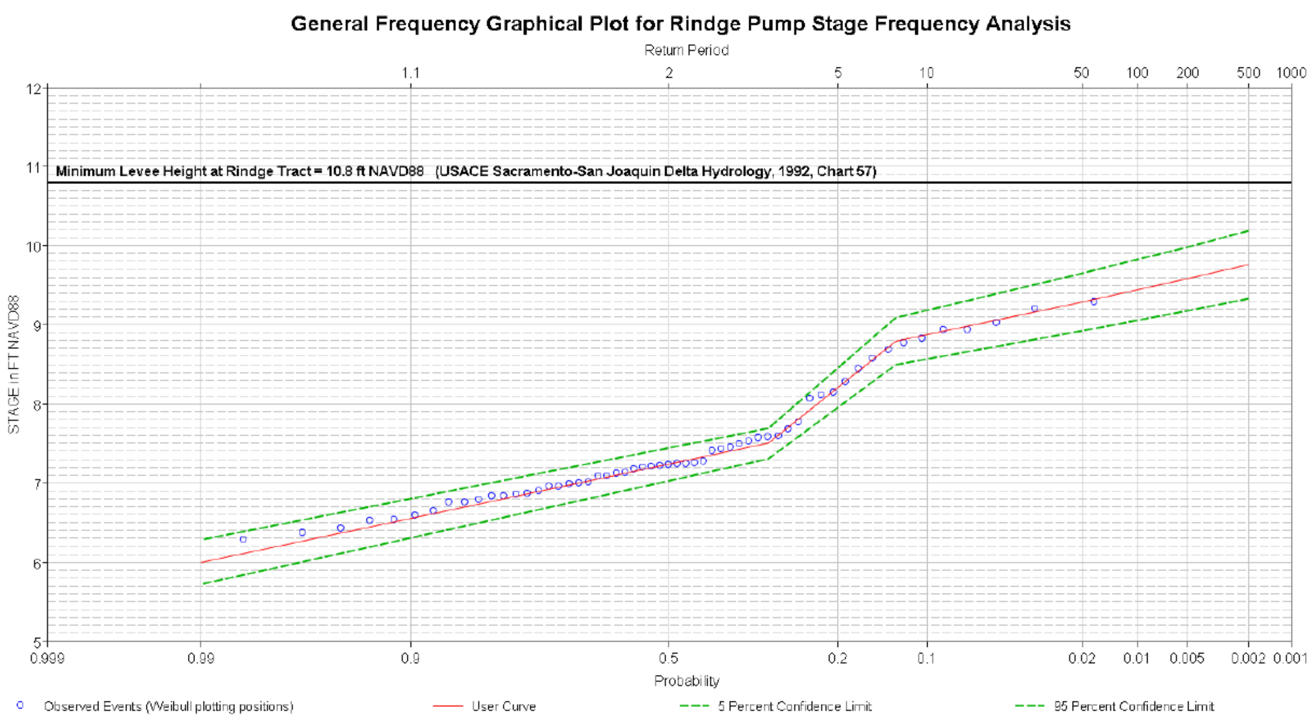


Plate 13b. General Frequency Graphical Plot Stage Frequency Analysis

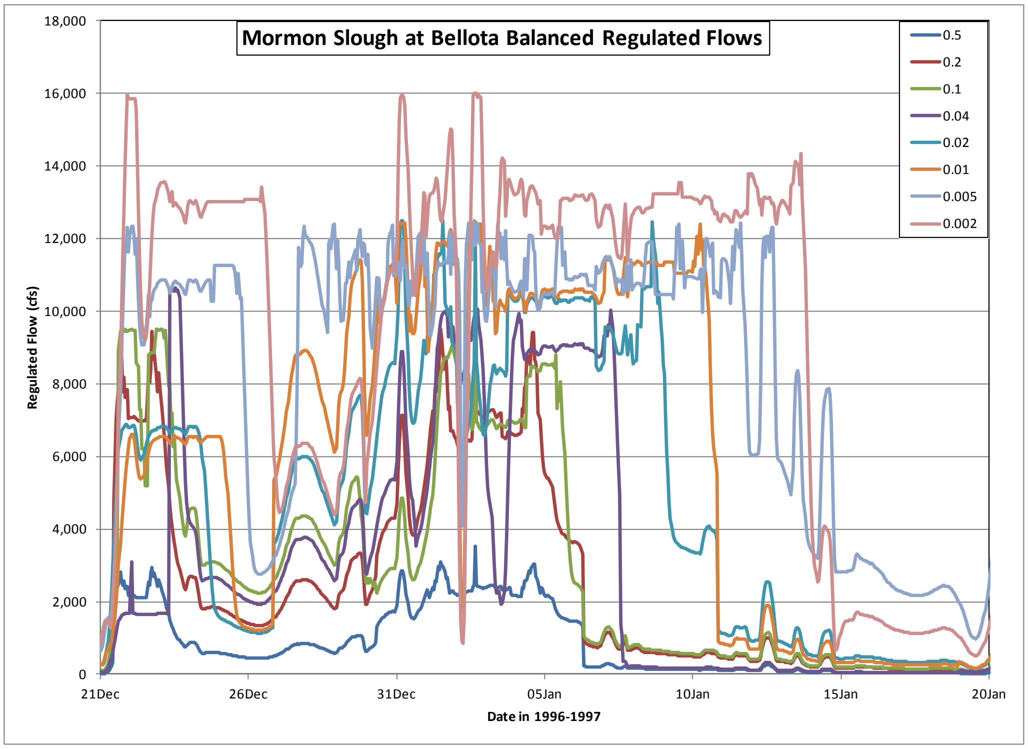


Plate 14. 0.5 to 0.002 AEP Regulated Hydrographs for the Calaveras River at Bellota

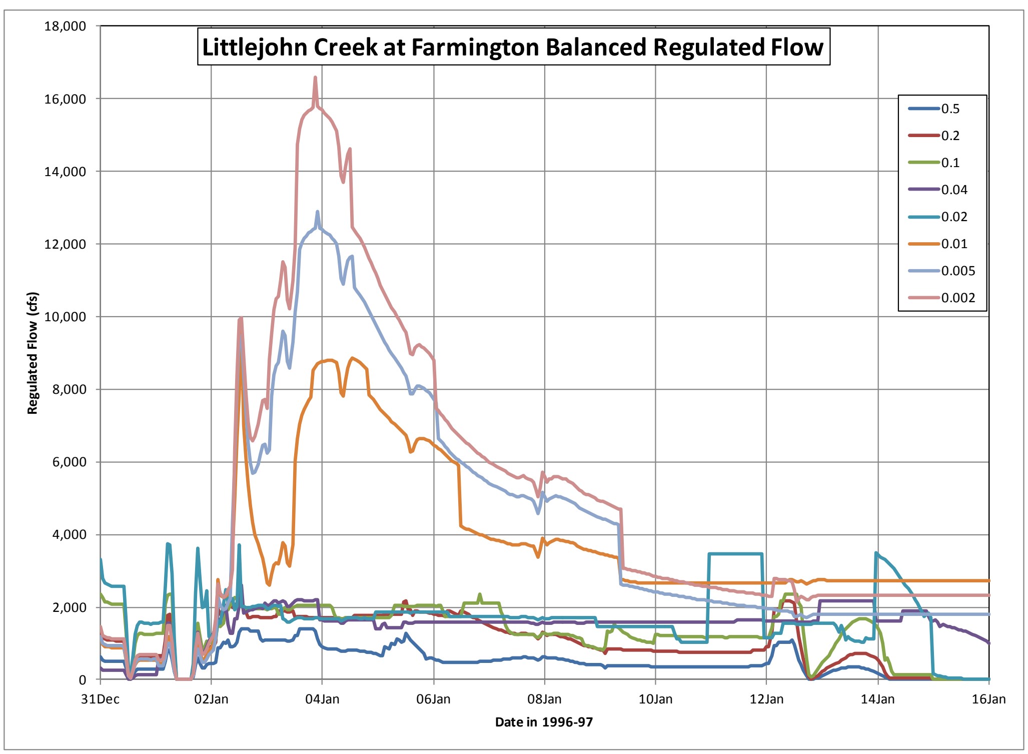


Plate 15. 0.5 to 0.002 AEP Regulated Hydrographs for Littlejohn Creek at Farmington

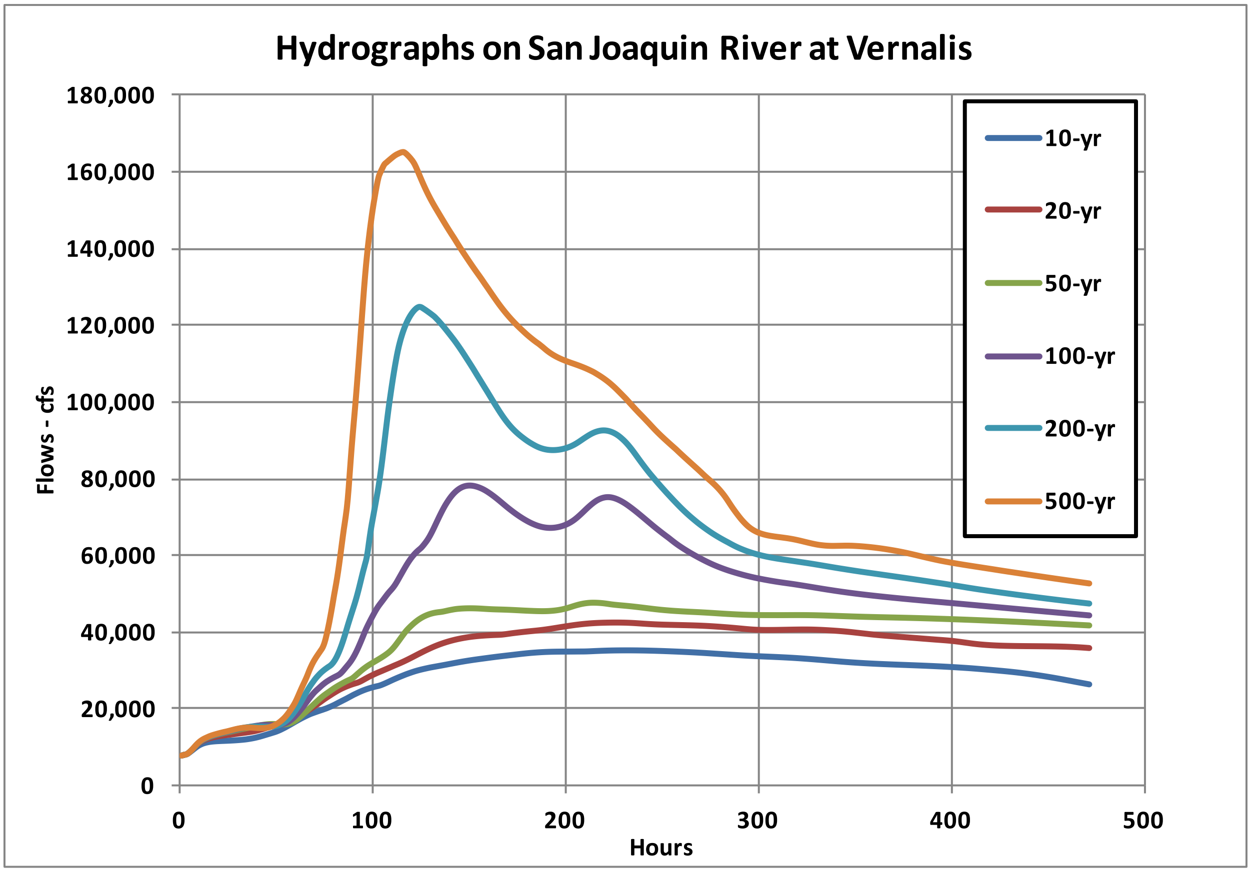
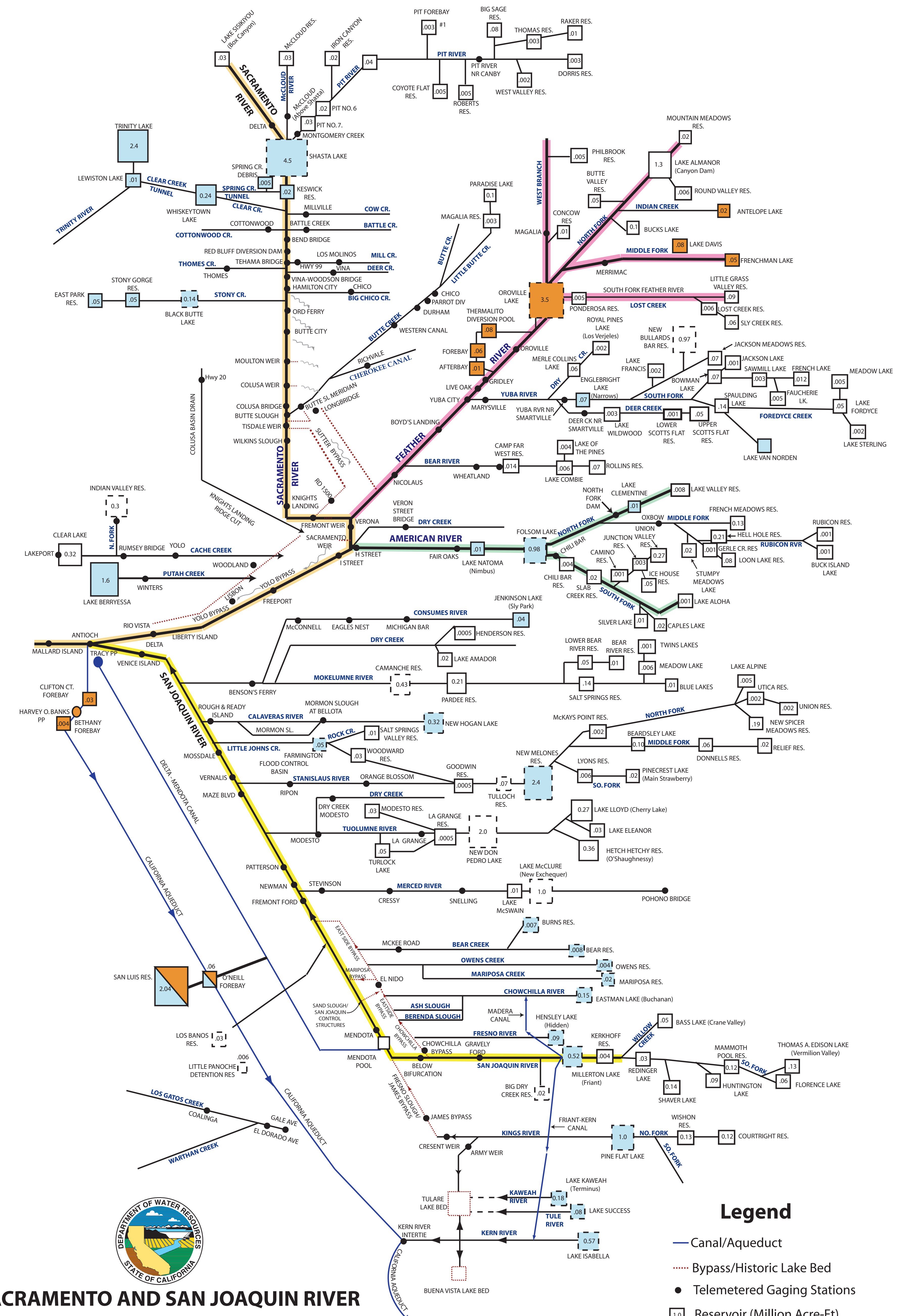


Plate 16. n-year Regulated Hydrographs for the San Joaquin River at Vernalis



SACRAMENTO AND SAN JOAQUIN RIVER SYSTEMS SCHEMATIC

Department of Water Resources
Division of Flood Management
(April 2012)

Legend

- Canal/Aqueduct
- Bypass/Historic Lake Bed
- Telemetered Gaging Stations
- Reservoir (Million Acre-Ft)
- State of California Owned
- Federally Owned
- Other Agency Owned
- Flood Control Reservoir (Million Acre-Ft)
- Floodway

Plate 17. San Joaquin River Basin Systems Schematic

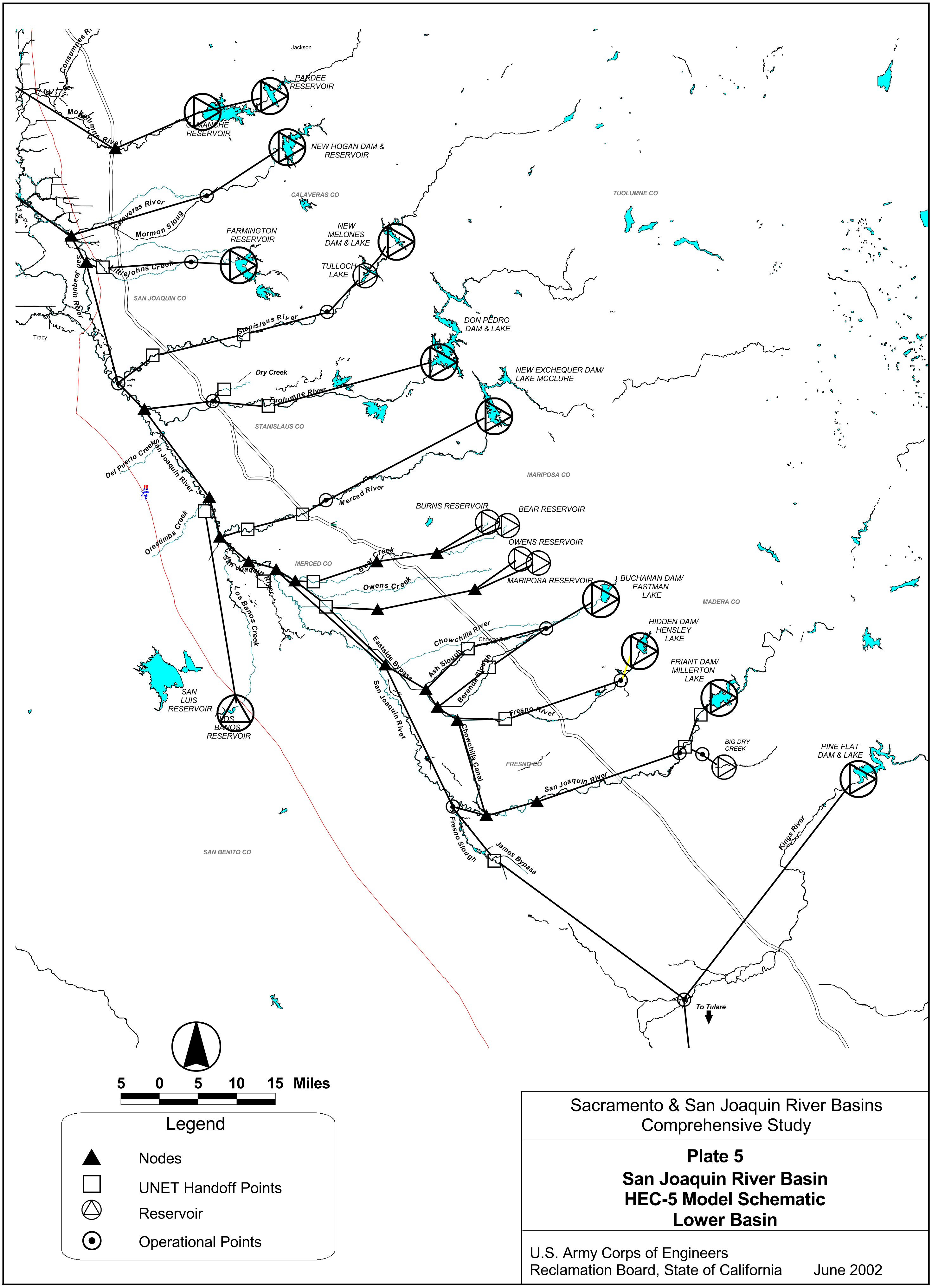
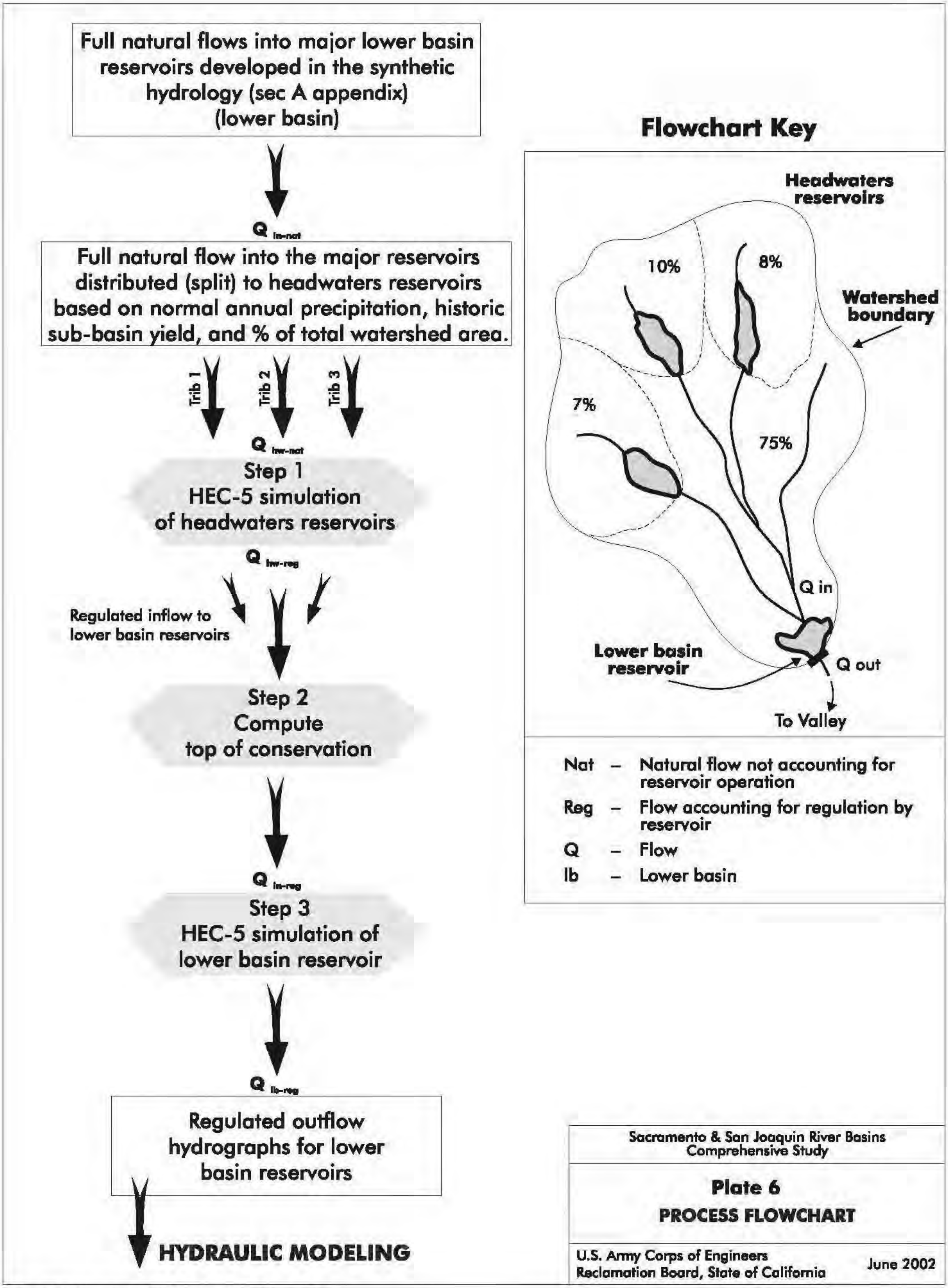


Plate 18. San Joaquin River Basin HEC-5 Model Schematic Lower Basin



US_Army_COE-Sac\comprehensive study\F4 conference\appendix figs june 02.cdr

Plate 19. Comp Study Process Flowchart

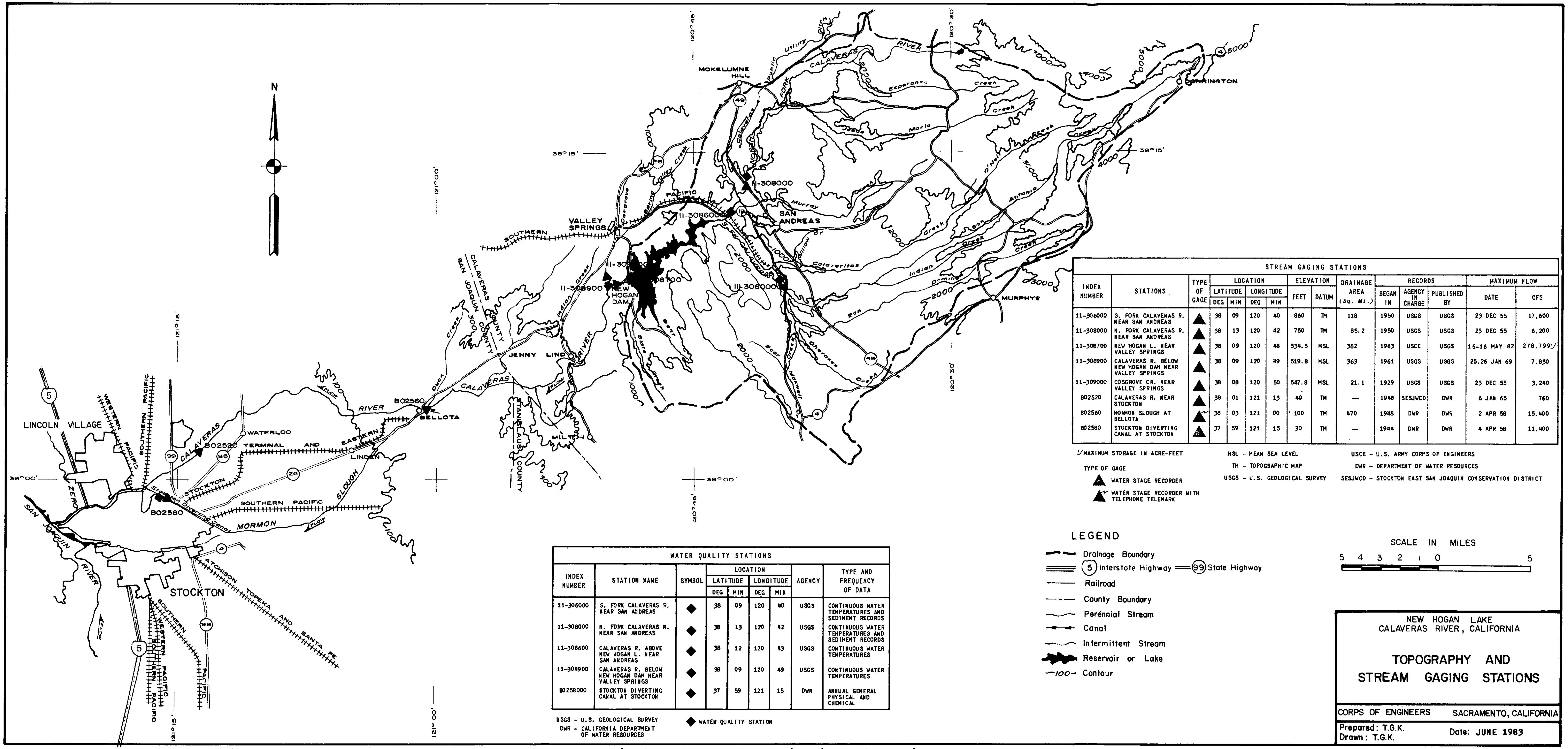
NEW HOGAN DAM AND LAKE
CALAVERAS RIVER,
CALIFORNIA

WATER CONTROL MANUAL

APPENDIX III TO
MASTER WATER CONTROL MANUAL
SAN JOAQUIN RIVER BASIN, CALIFORNIA

US ARMY CORPS
OF ENGINEERS
Sacramento District

JUNE 1983



STREAM GAGING STATIONS															
INDEX NUMBER	STATIONS	TYPE OF GAGE	LOCATION				ELEVATION		DRAINAGE AREA (Sq. Mi.)	RECORDS			MAXIMUM FLOW	
			LATITUDE		LONGITUDE		FEET	DATUM		BEGAN IN	AGENCY IN CHARGE	PUBLISHED BY	DATE	CFS
			DEG	MIN	DEG	MIN								
11-306000	S. FORK CALAVERAS R. NEAR SAN ANDREAS	▲	38	09	120	40	860	TM	118	1950	USGS	USGS	23 DEC 55	17,600
11-308000	N. FORK CALAVERAS R. NEAR SAN ANDREAS	▲	38	13	120	42	750	TM	85.2	1950	USGS	USGS	23 DEC 55	6,200
11-308700	NEW HOGAN L. NEAR VALLEY SPRINGS	▲	38	09	120	48	534.5	MSL	362	1963	USCE	USGS	15-16 MAY 82	278,799[1]
11-308900	CALAVERAS R. BELOW NEW HOGAN DAM NEAR VALLEY SPRINGS	▲	38	09	120	49	519.8	MSL	363	1961	USGS	USGS	25,26 JAN 69	7,830
11-309000	COSGROVE CR. NEAR VALLEY SPRINGS	▲	38	08	120	50	547.8	MSL	21.1	1929	USGS	USGS	23 DEC 55	3,240
B02520	CALAVERAS R. NEAR STOCKTON	▲	38	01	121	13	40	TM	—	1948	SESJWCD	DWR	6 JAN 65	760
B02560	MORMON SLOUGH AT BELLOTA	▲	38	03	121	00	100	TM	470	1948	DWR	DWR	2 APR 58	15,400
B02580	STOCKTON DIVERTING CANAL AT STOCKTON	▲	37	59	121	15	30	TM	—	1944	DWR	DWR	4 APR 58	11,400

[1] MAXIMUM STORAGE IN ACRE-FEET

TYPE OF GAGE

▲ WATER STAGE RECORDER

▲ WATER STAGE RECORDER WITH TELEPHONE TELEMARK

MSL - MEAN SEA LEVEL

TM - TOPOGRAPHIC MAP

USGS - U.S. GEOLOGICAL SURVEY

USCE - U.S. ARMY CORPS OF ENGINEERS

DWR - DEPARTMENT OF WATER RESOURCES

SESJWCD - STOCKTON EAST SAN JOAQUIN CONSERVATION DISTRICT

WATER QUALITY STATIONS								
INDEX NUMBER	STATION NAME	SYMBOL	LOCATION				AGENCY	TYPE AND FREQUENCY OF DATA
			LATITUDE		LONGITUDE			
			DEG	MIN	DEG	MIN		
11-306000	S. FORK CALAVERAS R. NEAR SAN ANDREAS	◆	38	09	120	40	USGS	CONTINUOUS WATER TEMPERATURES AND SEDIMENT RECORDS
11-308000	N. FORK CALAVERAS R. NEAR SAN ANDREAS	◆	38	13	120	42	USGS	CONTINUOUS WATER TEMPERATURES AND SEDIMENT RECORDS
11-308600	CALAVERAS R. ABOVE NEW HOGAN L. NEAR SAN ANDREAS	◆	38	12	120	43	USGS	CONTINUOUS WATER TEMPERATURES
11-308900	CALAVERAS R. BELOW NEW HOGAN DAM NEAR VALLEY SPRINGS	◆	38	09	120	49	USGS	CONTINUOUS WATER TEMPERATURES
B0258000	STOCKTON DIVERTING CANAL AT STOCKTON	◆	37	59	121	15	DWR	ANNUAL GENERAL PHYSICAL AND CHEMICAL

USGS - U.S. GEOLOGICAL SURVEY

DWR - CALIFORNIA DEPARTMENT OF WATER RESOURCES

◆ WATER QUALITY STATION

LEGEND

- Drainage Boundary
- (5) Interstate Highway (99) State Highway
- Railroad
- County Boundary
- Perennial Stream
- Canal
- Intermittent Stream
- Reservoir or Lake
- 100 Contour

SCALE IN MILES

5 4 3 2 1 0 5

NEW HOGAN LAKE
CALAVERAS RIVER, CALIFORNIA

TOPOGRAPHY AND STREAM GAGING STATIONS

CORPS OF ENGINEERS — SACRAMENTO, CALIFORNIA

Prepared: T.G.K.
Drawn: T.G.K.

Date: JUNE 1983

Plate 20. New Hogan Dam Topography and Stream Gage Stations

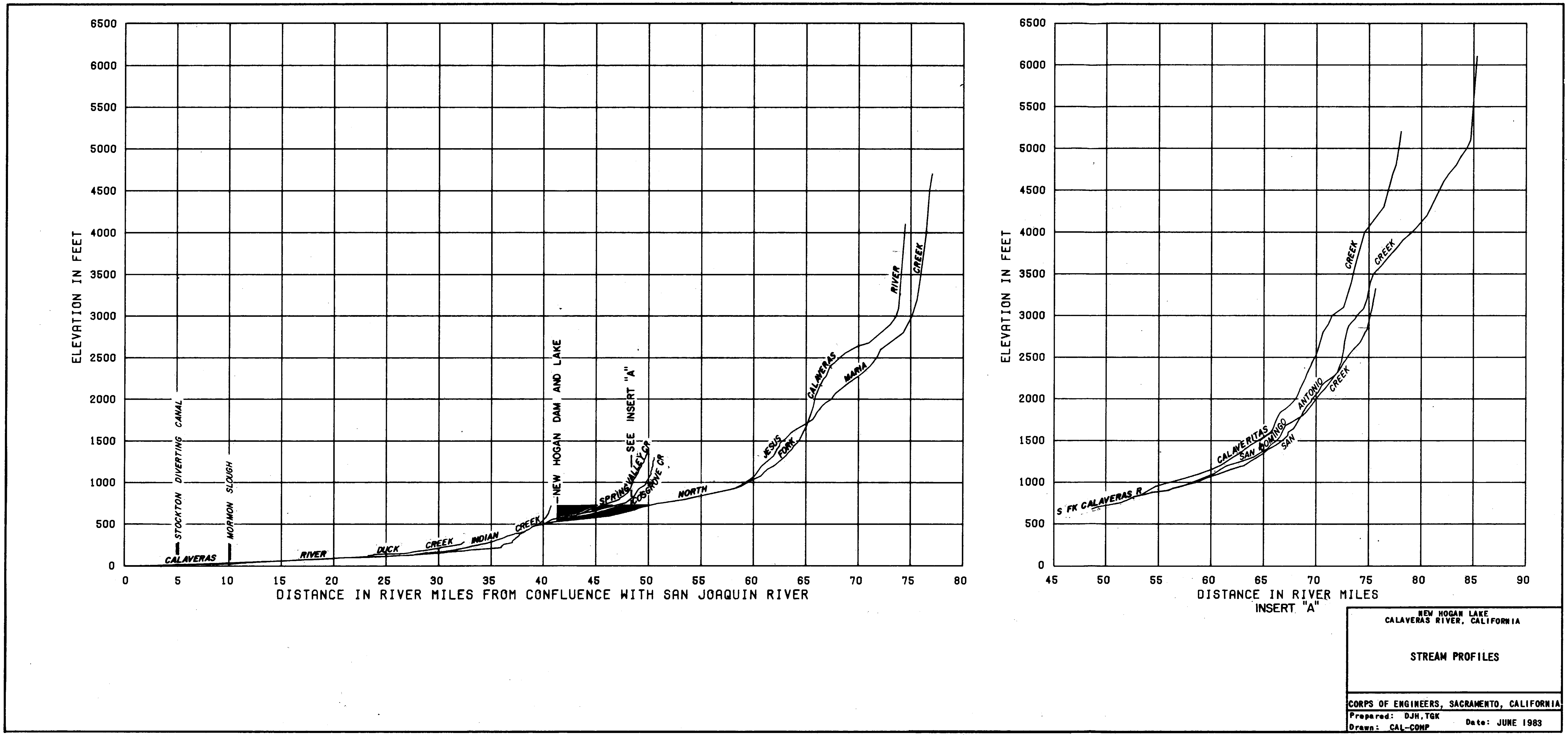
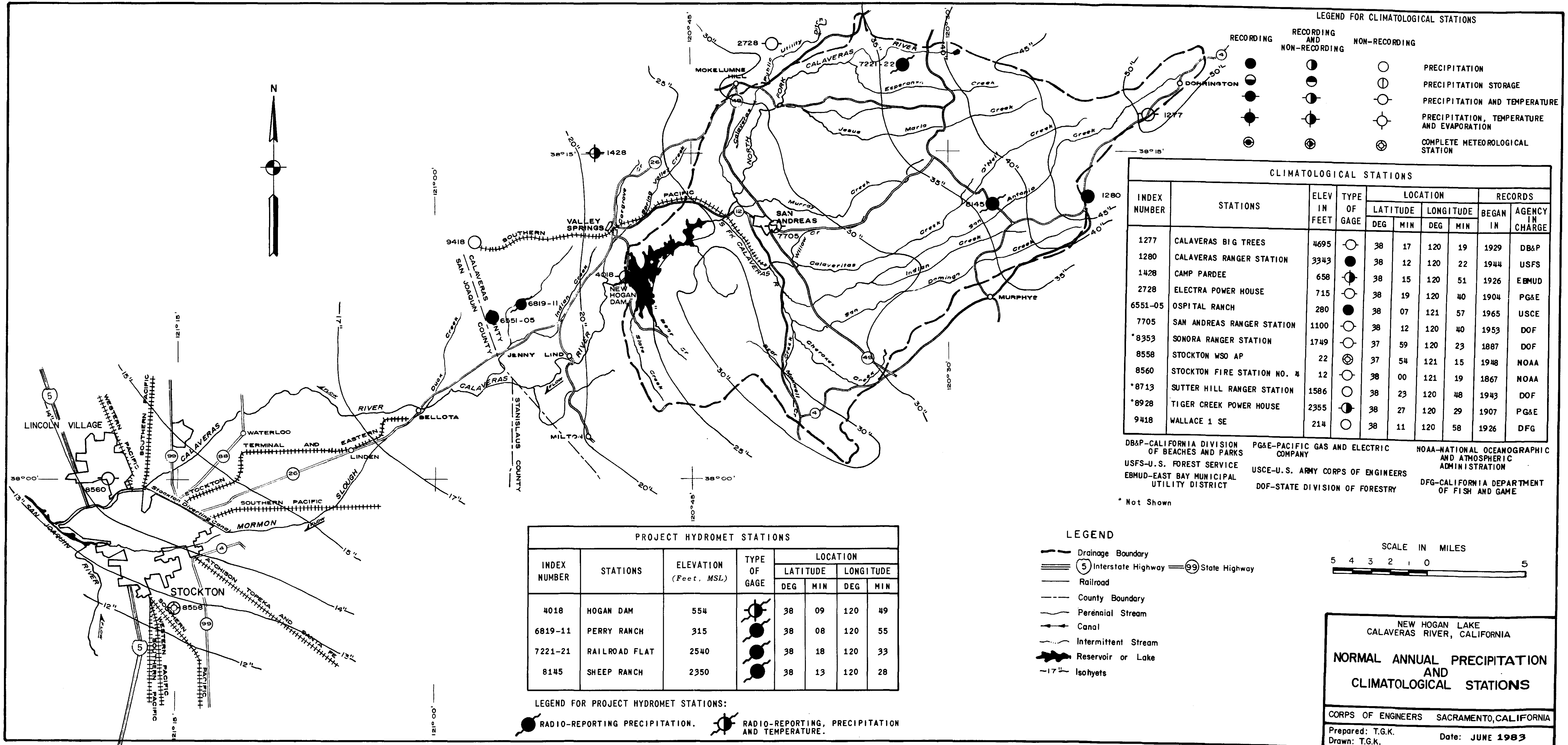


Plate 21. New Hogan Dam Stream Profiles

PLATE 11



LEGEND FOR CLIMATOLOGICAL STATIONS

RECORDING — RECORDING AND NON-RECORDING — NON-RECORDING

PRECIPITATION
PRECIPITATION STORAGE
PRECIPITATION AND TEMPERATURE
PRECIPITATION, TEMPERATURE AND EVAPORATION
COMPLETE METEOROLOGICAL STATION

CLIMATOLOGICAL STATIONS

INDEX NUMBER	STATIONS	ELEV IN FEET	TYPE OF GAGE	LOCATION				RECORDS	
				LATITUDE		LONGITUDE		BEGAN IN	AGENCY IN CHARGE
				DEG	MIN	DEG	MIN		
1277	CALAVERAS BIG TREES	4695		38	17	120	19	1929	DB&P
1280	CALAVERAS RANGER STATION	3343		38	12	120	22	1944	USFS
1428	CAMP PARDEE	658		38	15	120	51	1926	EBMUD
2728	ELECTRA POWER HOUSE	715		38	19	120	40	1904	PG&E
6551-05	OSPITAL RANCH	280		38	07	121	57	1965	USCE
7705	SAN ANDREAS RANGER STATION	1100		38	12	120	40	1953	DOF
*8353	SONORA RANGER STATION	1749		37	59	120	23	1887	DOF
8558	STOCKTON WSO AP	22		37	54	121	15	1948	NOAA
8560	STOCKTON FIRE STATION NO. 4	12		38	00	121	19	1867	NOAA
*8713	SUTTER HILL RANGER STATION	1586		38	23	120	48	1943	DOF
*8928	TIGER CREEK POWER HOUSE	2355		38	27	120	29	1907	PG&E
9418	WALLACE 1 SE	214		38	11	120	58	1926	DFG

DB&P–CALIFORNIA DIVISION OF BEACHES AND PARKS
USFS–U.S. FOREST SERVICE
EBMUD–EAST BAY MUNICIPAL UTILITY DISTRICT
PG&E–PACIFIC GAS AND ELECTRIC COMPANY
USCE–U.S. ARMY CORPS OF ENGINEERS
DOF–STATE DIVISION OF FORESTRY
NOAA–NATIONAL OCEANOGRAPHIC AND ATMOSPHERIC ADMINISTRATION
DFG–CALIFORNIA DEPARTMENT OF FISH AND GAME

* Not Shown

PROJECT HYDROMET STATIONS

INDEX NUMBER	STATIONS	ELEVATION (Feet, MSL)	TYPE OF GAGE	LOCATION			
				LATITUDE		LONGITUDE	
				DEG	MIN	DEG	MIN
4018	HOGAN DAM	554		38	09	120	49
6819-11	PERRY RANCH	315		38	08	120	55
7221-21	RAILROAD FLAT	2540		38	18	120	33
8145	SHEEP RANCH	2350		38	13	120	28

LEGEND FOR PROJECT HYDROMET STATIONS:

RADIO-REPORTING PRECIPITATION.
RADIO-REPORTING, PRECIPITATION AND TEMPERATURE.

LEGEND

Drainage Boundary
5 Interstate Highway
99 State Highway
Railroad
County Boundary
Perennial Stream
Canal
Intermittent Stream
Reservoir or Lake
—17"— Isohyets

SCALE IN MILES

5 4 3 2 1 0 5

NEW HOGAN LAKE
CALAVERAS RIVER, CALIFORNIA

NORMAL ANNUAL PRECIPITATION AND CLIMATOLOGICAL STATIONS

CORPS OF ENGINEERS SACRAMENTO, CALIFORNIA

Prepared: T.G.K.
Drawn: T.G.K.
Date: JUNE 1983

Plate 22. New Hogan Dam NAP and Climate Stations

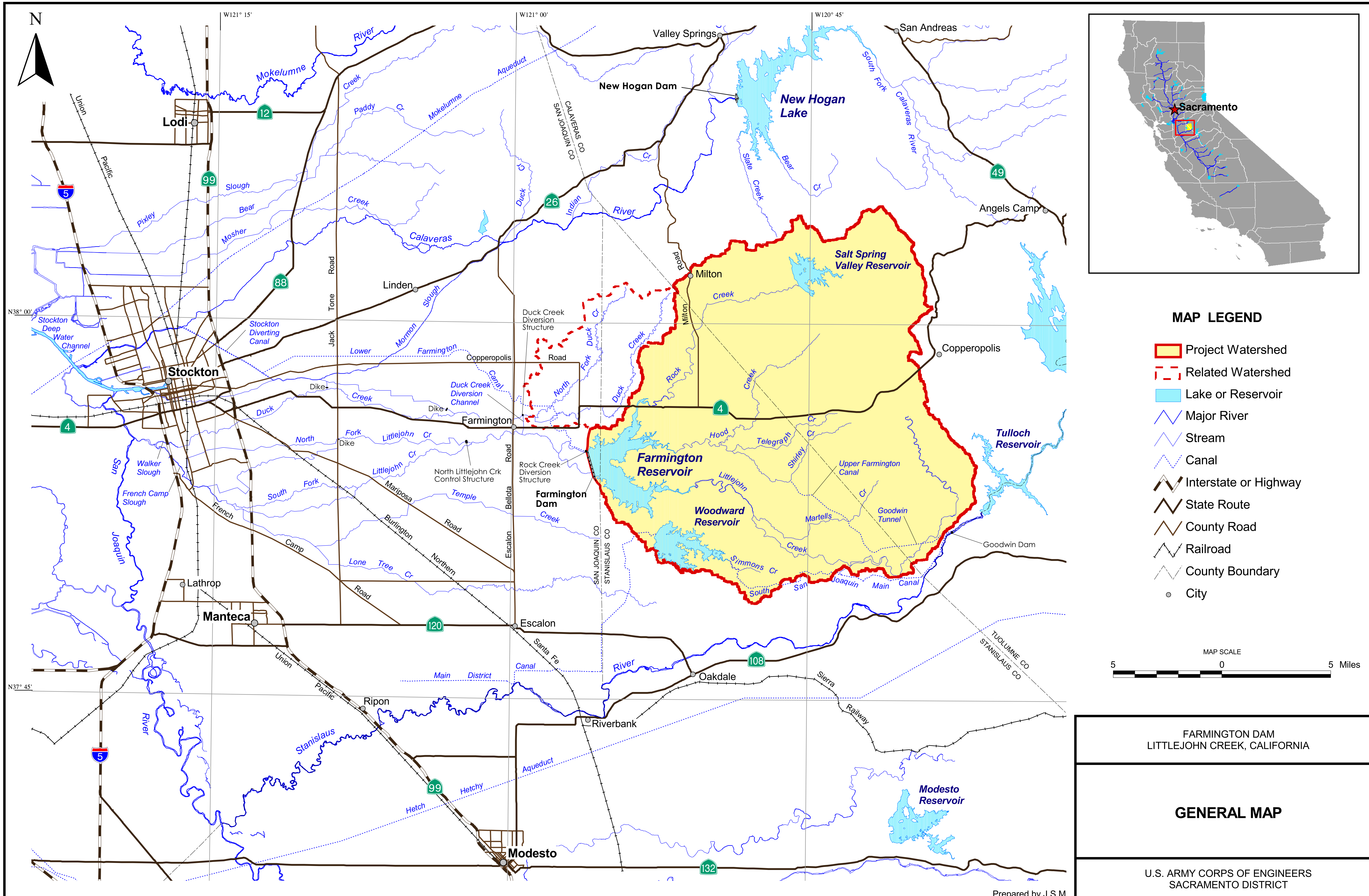
FARMINGTON
DAM AND RESERVOIR
LITTLEJOHN CREEK, CALIFORNIA

WATER CONTROL MANUAL

APPENDIX IV TO
MASTER WATER CONTROL MANUAL
SAN JOAQUIN RIVER BASIN, CALIFORNIA

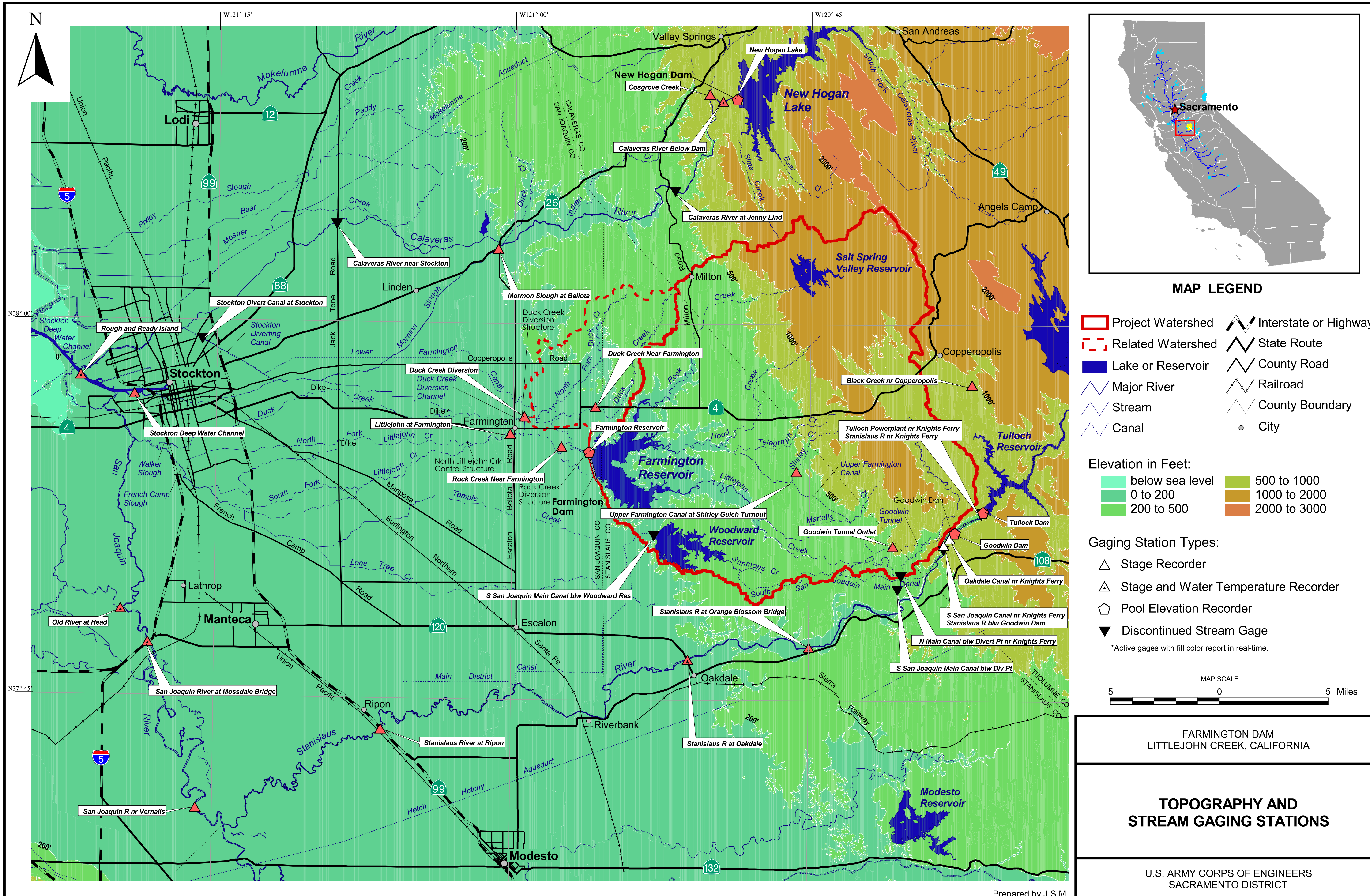
US ARMY CORPS
OF ENGINEERS
Sacramento District

DECEMBER 1952
REVISED DECEMBER 2004



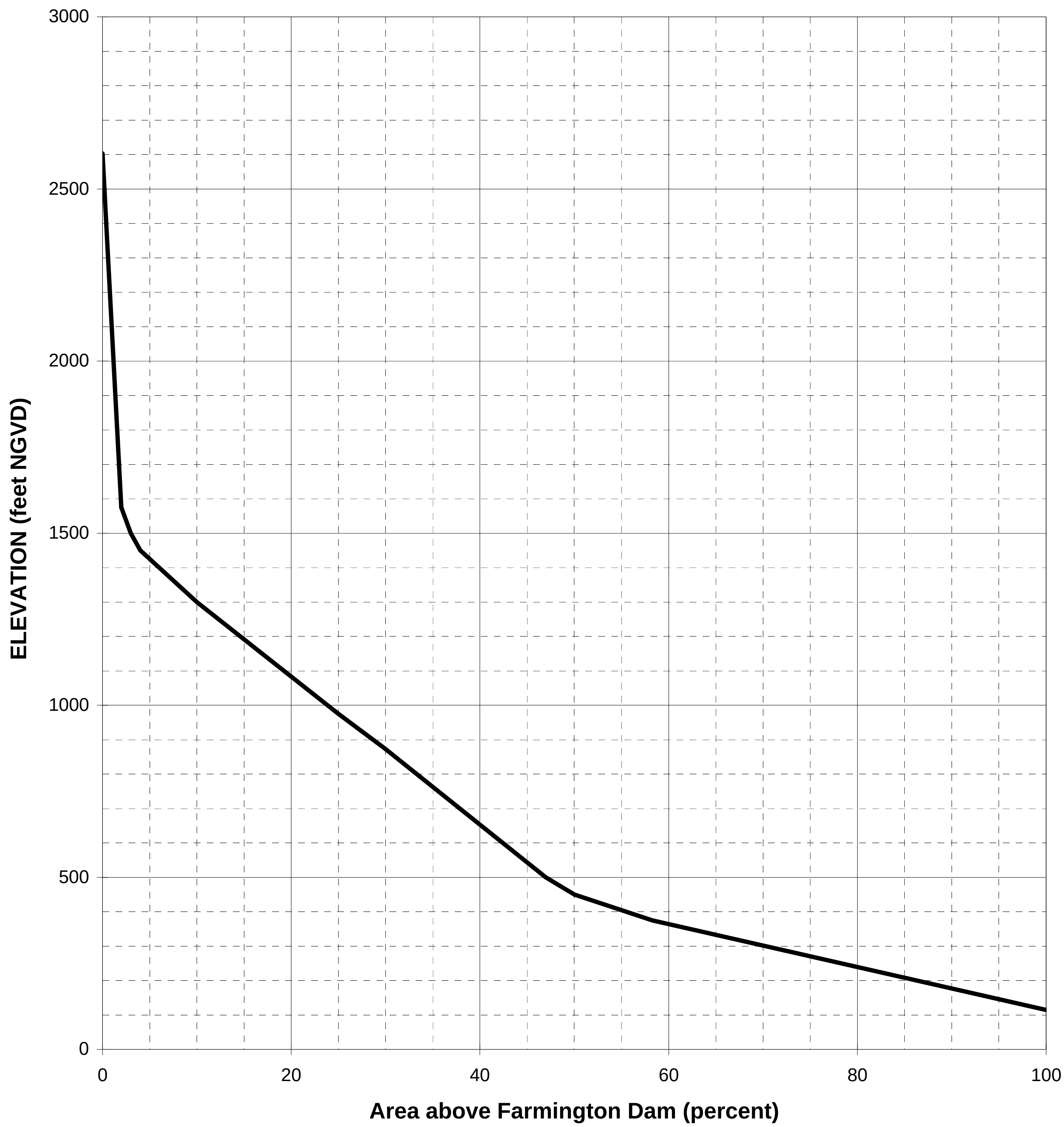
Revised Dec 2004 SHEET 1 OF 2 PLATE 2-1

Plate 23. Farmington Dam General Map



Revised Dec 2004 SHEET 1 OF 2 PLATE 4-1

Plate 24. Farmington Dam Topography and Stream Gaging Stations



NOTES:
1. Area: 212 square miles
2. Dam site elevation: 115 feet

Prepared by MVB

FARMINGTON DAM
LITTLEJOHN CREEK, CALIFORNIA

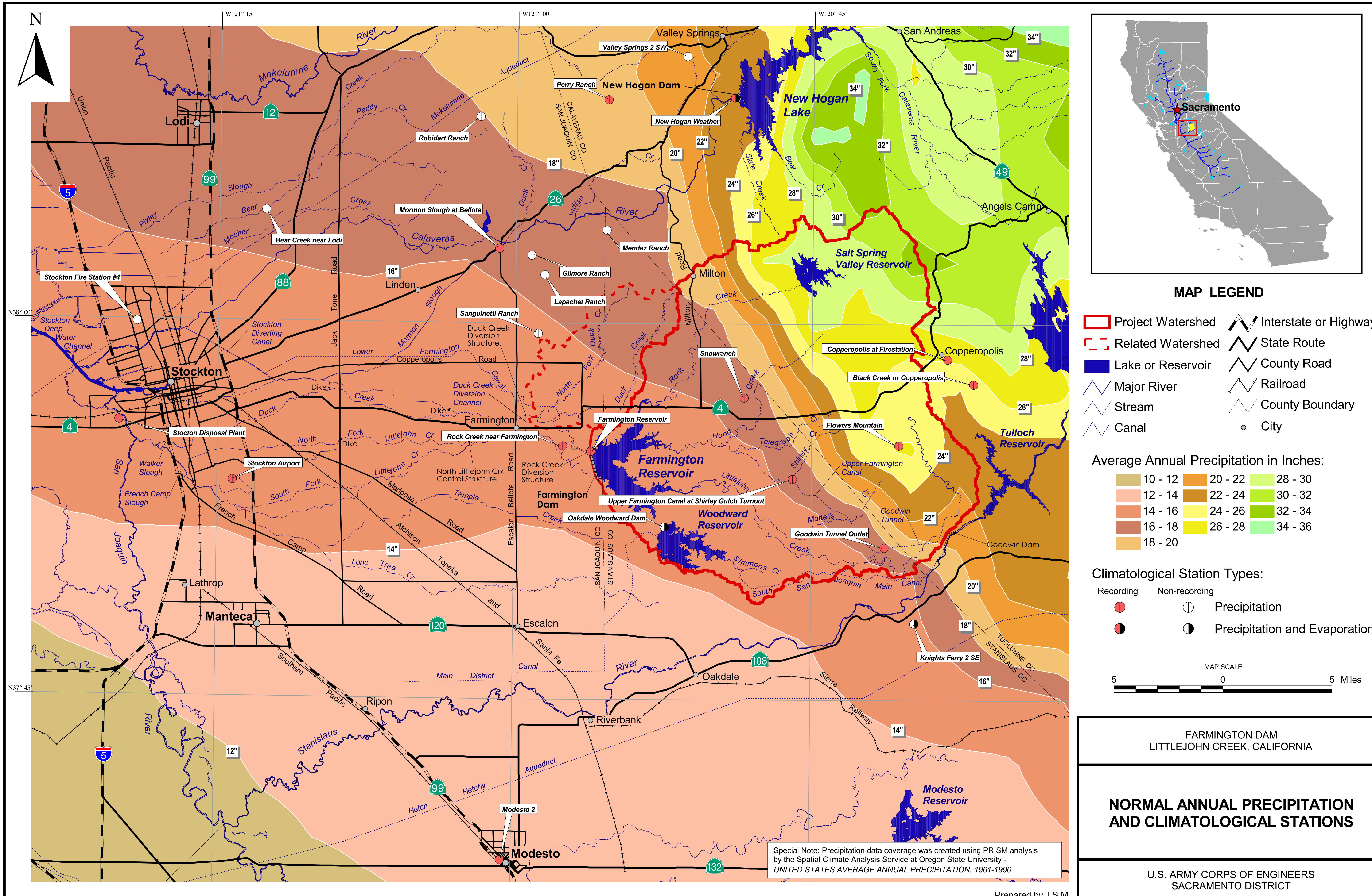
AREA-ELEVATION CURVE

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Revised Dec 2004

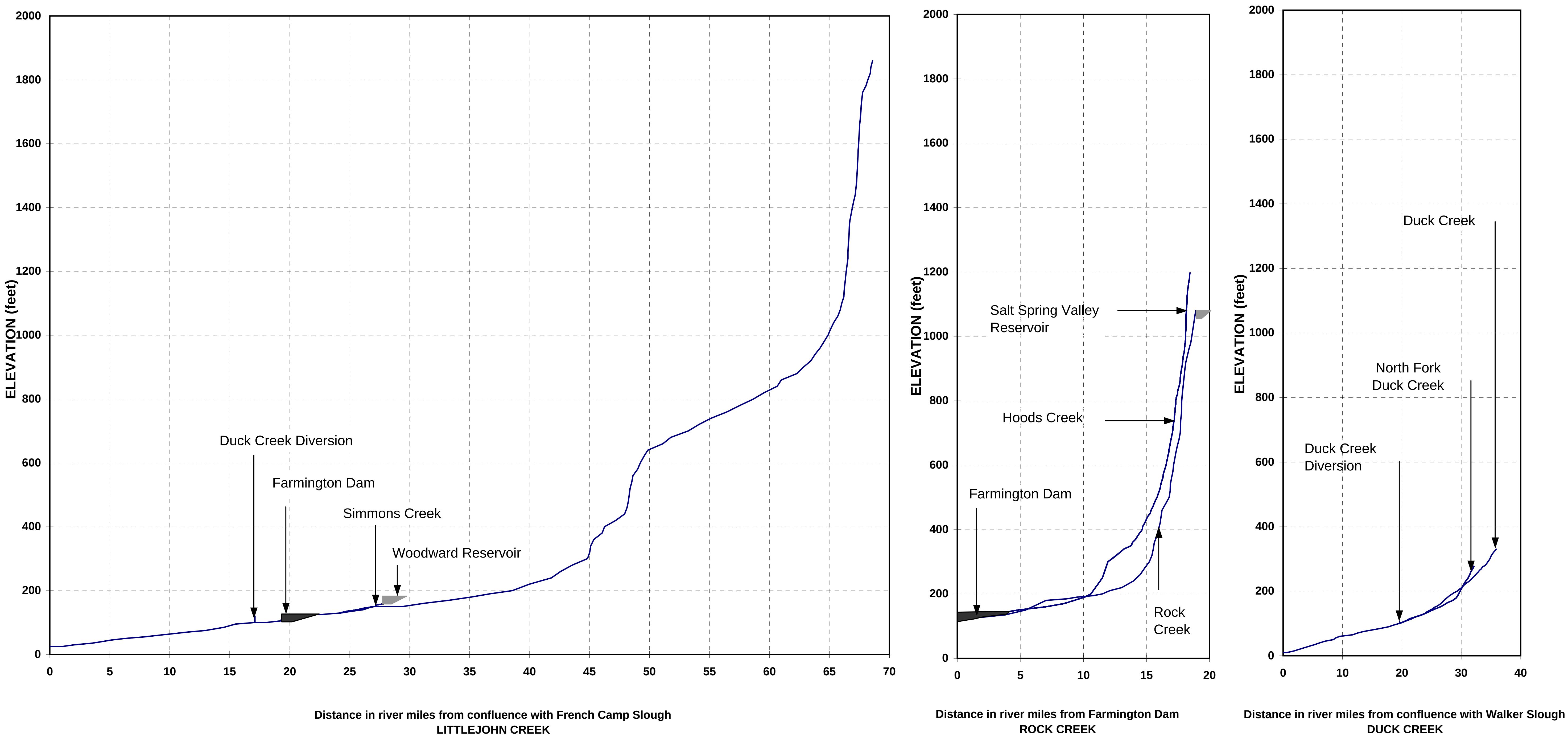
PLATE 4-3

Plate 25. Farmington Dam Area-Elevation Curve



Revised Dec 2004 SHEET 1 OF 2 PLATE 4-5

Plate 26. Farmington Dam NAP and Climate Stations



Prepared by MVB

FARMINGTON DAM
LITTLEJOHN CREEK, CALIFORNIA

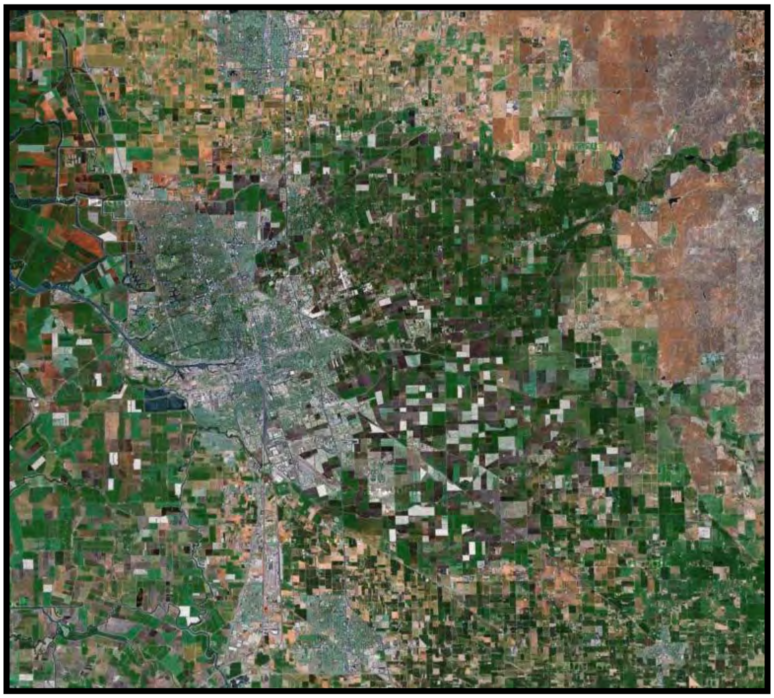
STREAM PROFILES

U.S. ARMY CORPS OF ENGINEERS
SACRAMENTO DISTRICT

Revised Dec 2004 PLATE 4-2

Plate 27. Farmington Dam Stream Profiles

LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY



F3 HYDROLOGY APPENDIX

JULY 30, 2012

US Army Corps of Engineers

SJAFCA San Joaquin Area FLOOD CONTROL Agency

DEPARTMENT OF WATER RESOURCES STATE OF CALIFORNIA

PREPARED BY:

PETERSON . BRUSTAD . INC
ENGINEERING . CONSULTING

1180 Iron Point Rd., Suite 260, Folsom, CA 95630
(916)608-2212

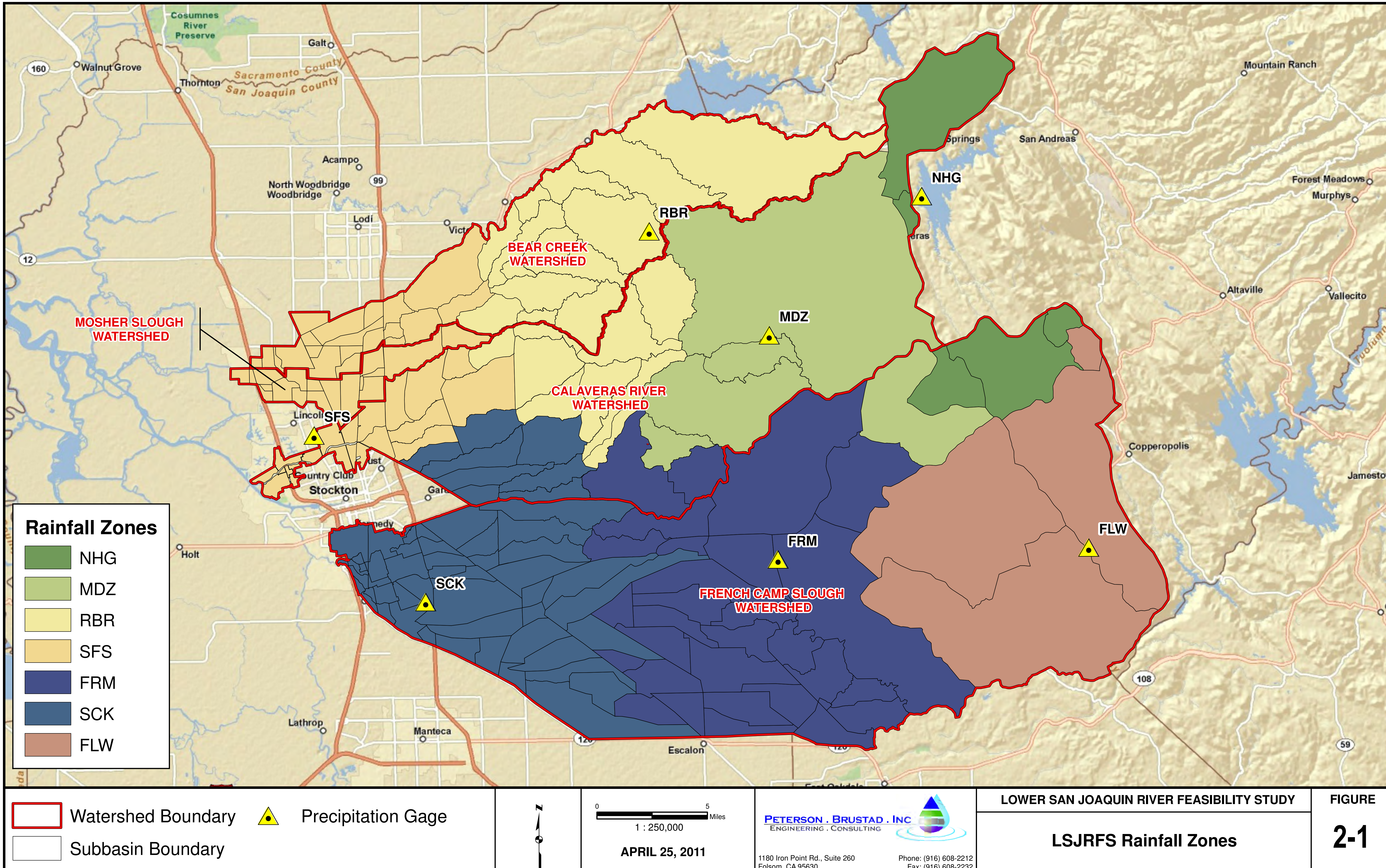


Plate 28. LSJRFS Rainfall Zones

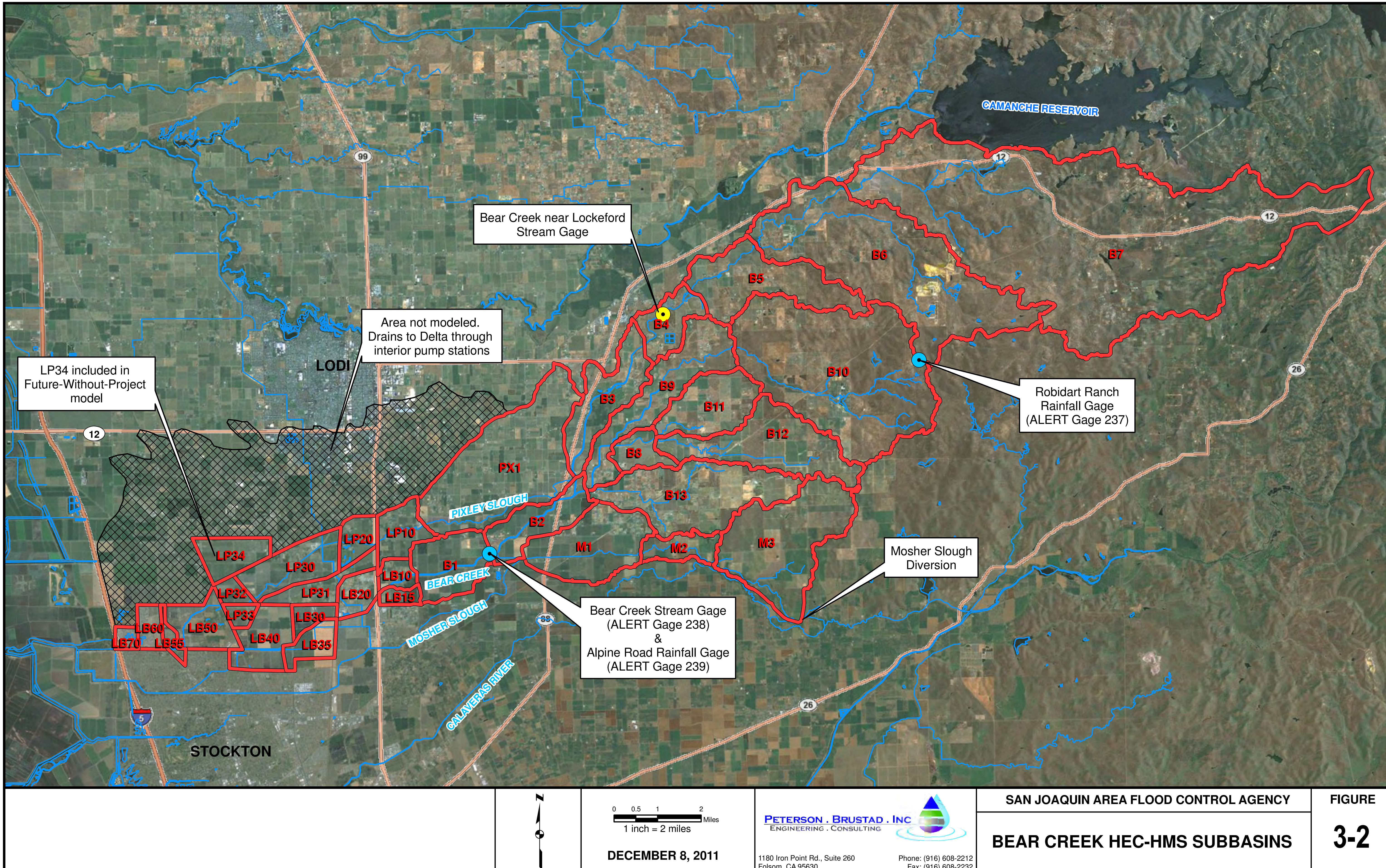
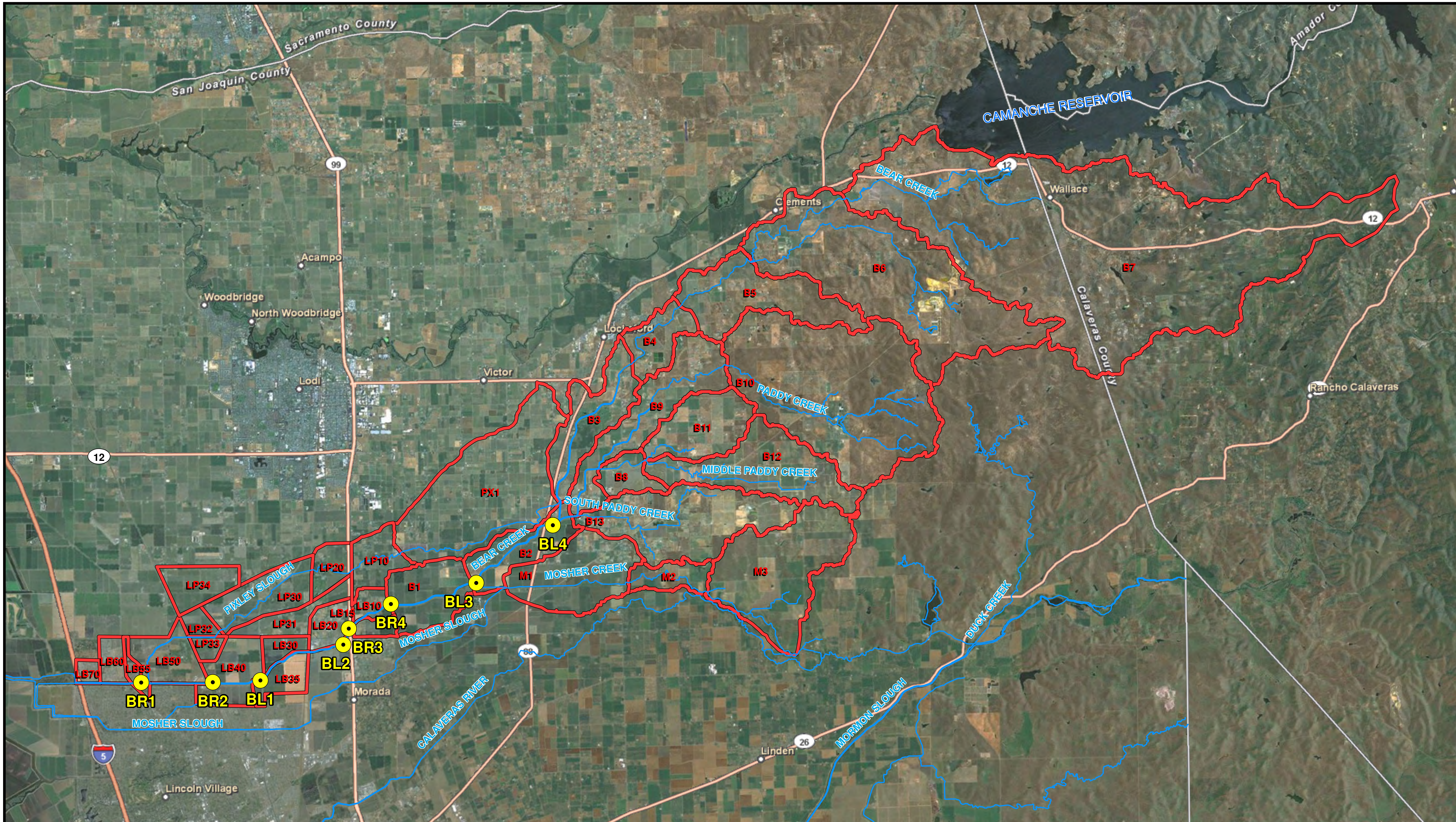


Plate 29. Bear Creek HEC-HMS Subbasins



LSJRFS Index Point

Subshed Boundary

N

0 0.5 1 2 Miles

1 inch = 2 miles

JUNE 20, 2012

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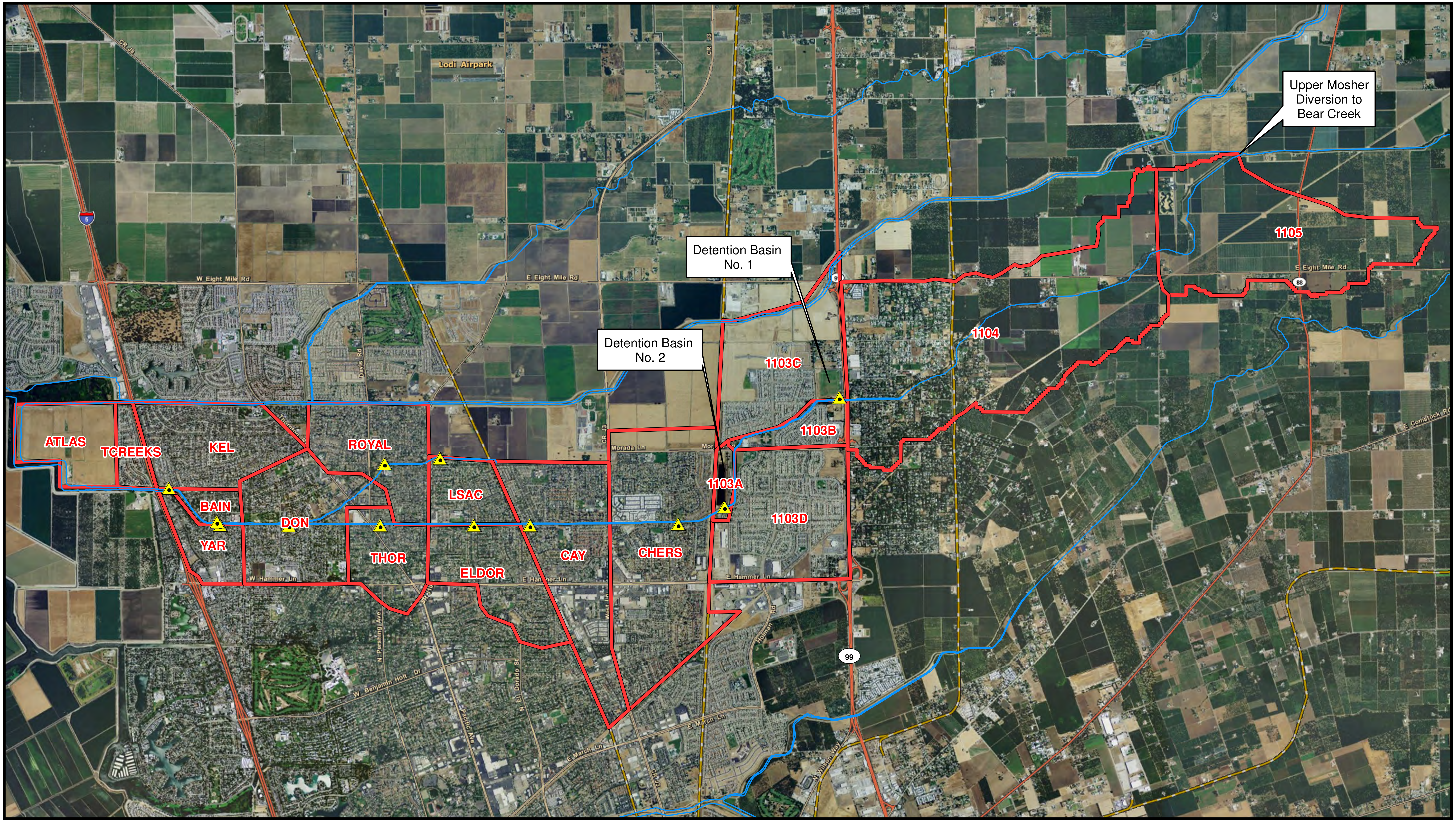
LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

BEAR CREEK WATERSHED
INDEX POINTS

FIGURE

3-12

Plate 30. Bear Creek Watershed Index Points



Subbasin Boundary

Existing Pump Station

N

0 1,000 2,000 4,000 Feet

1 inch = 4,000 feet

AUGUST 20, 2010

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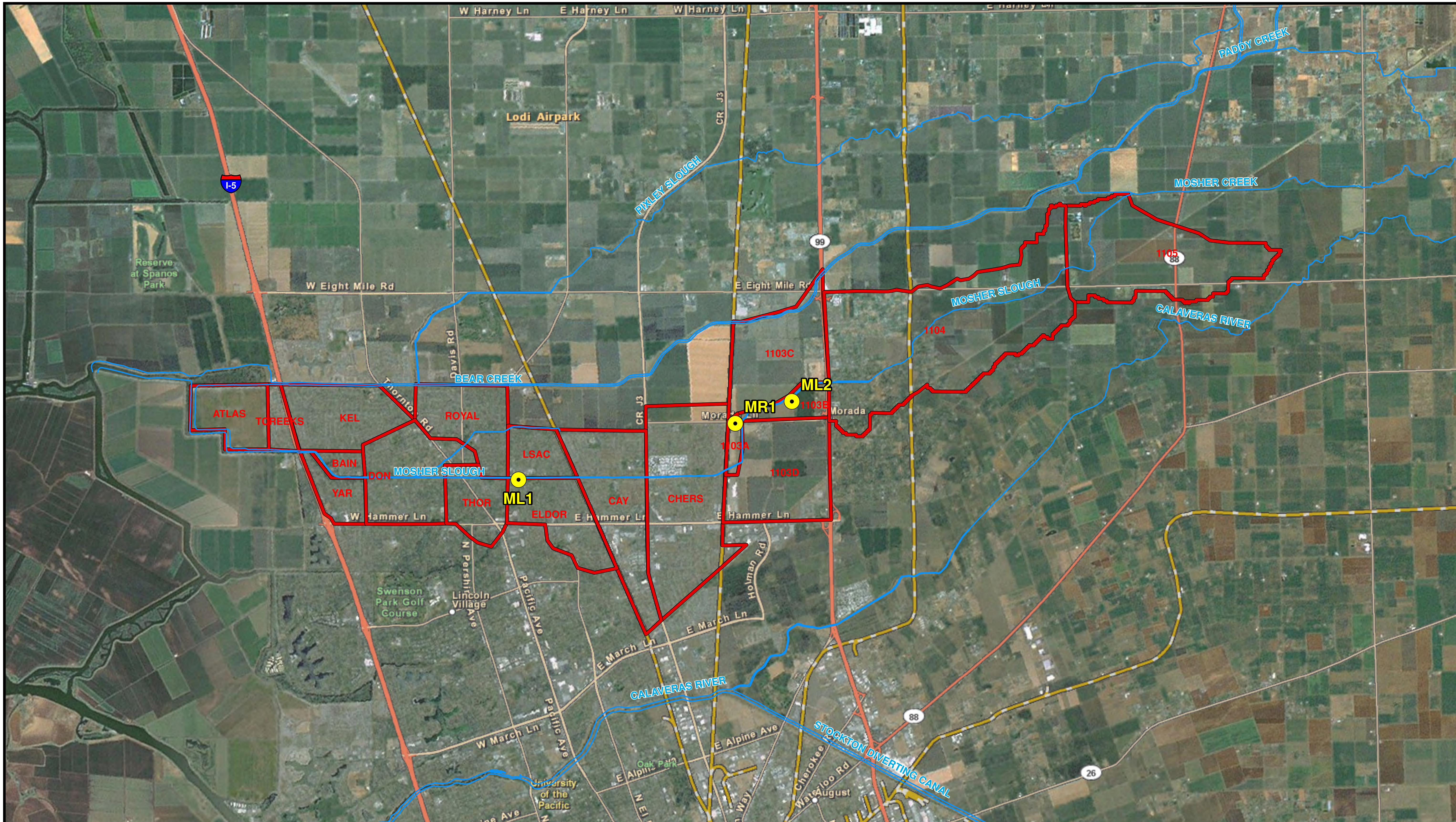
SAN JOAQUIN AREA FLOOD CONTROL AGENCY

MOSHER SLOUGH
HEC-HMS SUBBASINS

FIGURE

4-2

Plate 31. Mosher Slough HEC-HMS Subbasins



LSJRFS Index Point

Subshed Boundary

0 0.25 0.5 1 Miles

1 inch = 1 mile

JUNE 20, 2012

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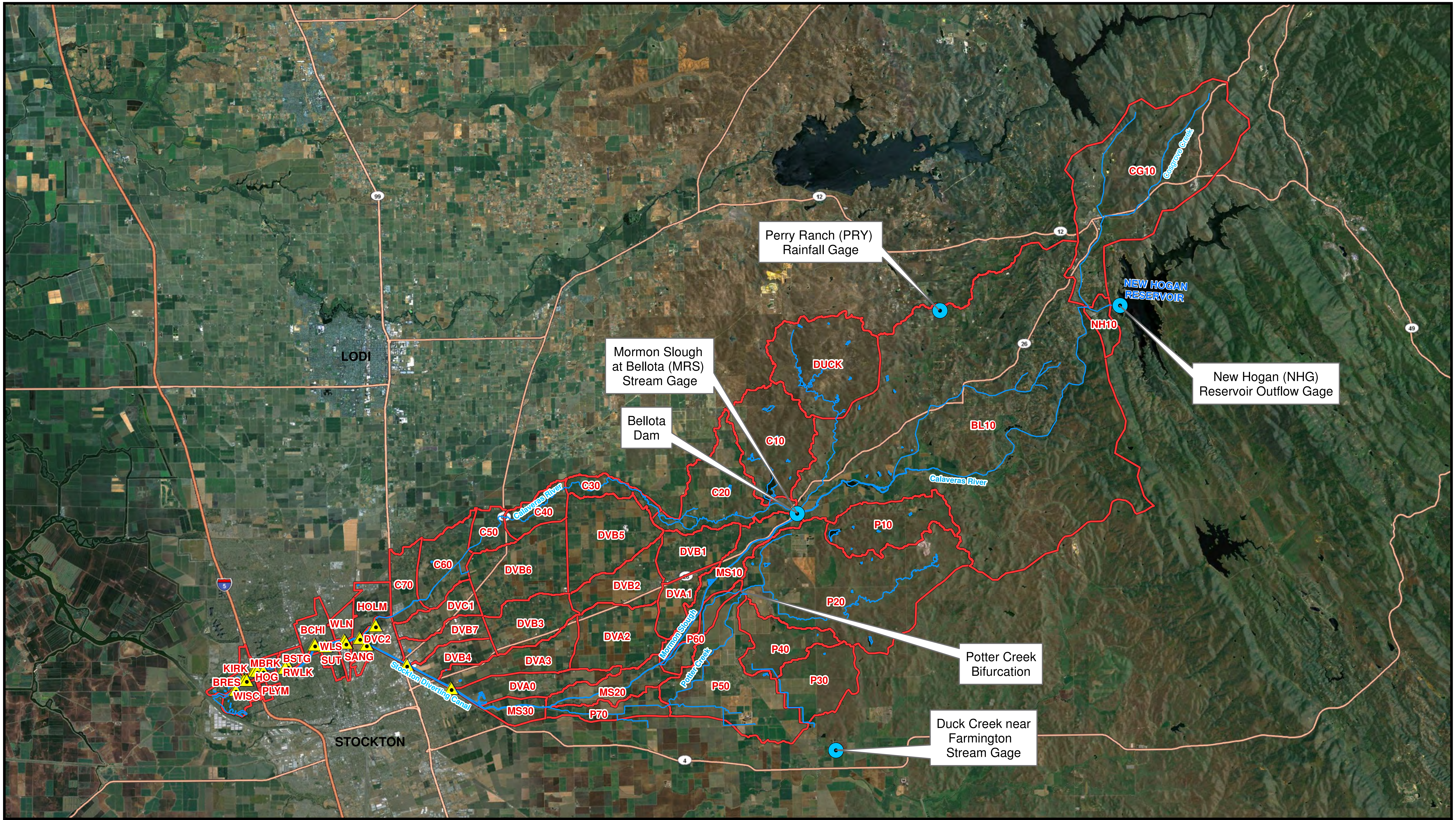
LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

MOSHER SLOUGH WATERSHED INDEX POINTS

FIGURE

4-10

Plate 32. Mosher Slough Watershed Index Points



Subbasin Boundary

Existing Pump Station

0 1.5 3 Miles

1 inch = 3 miles

SEPTEMBER 21, 2010

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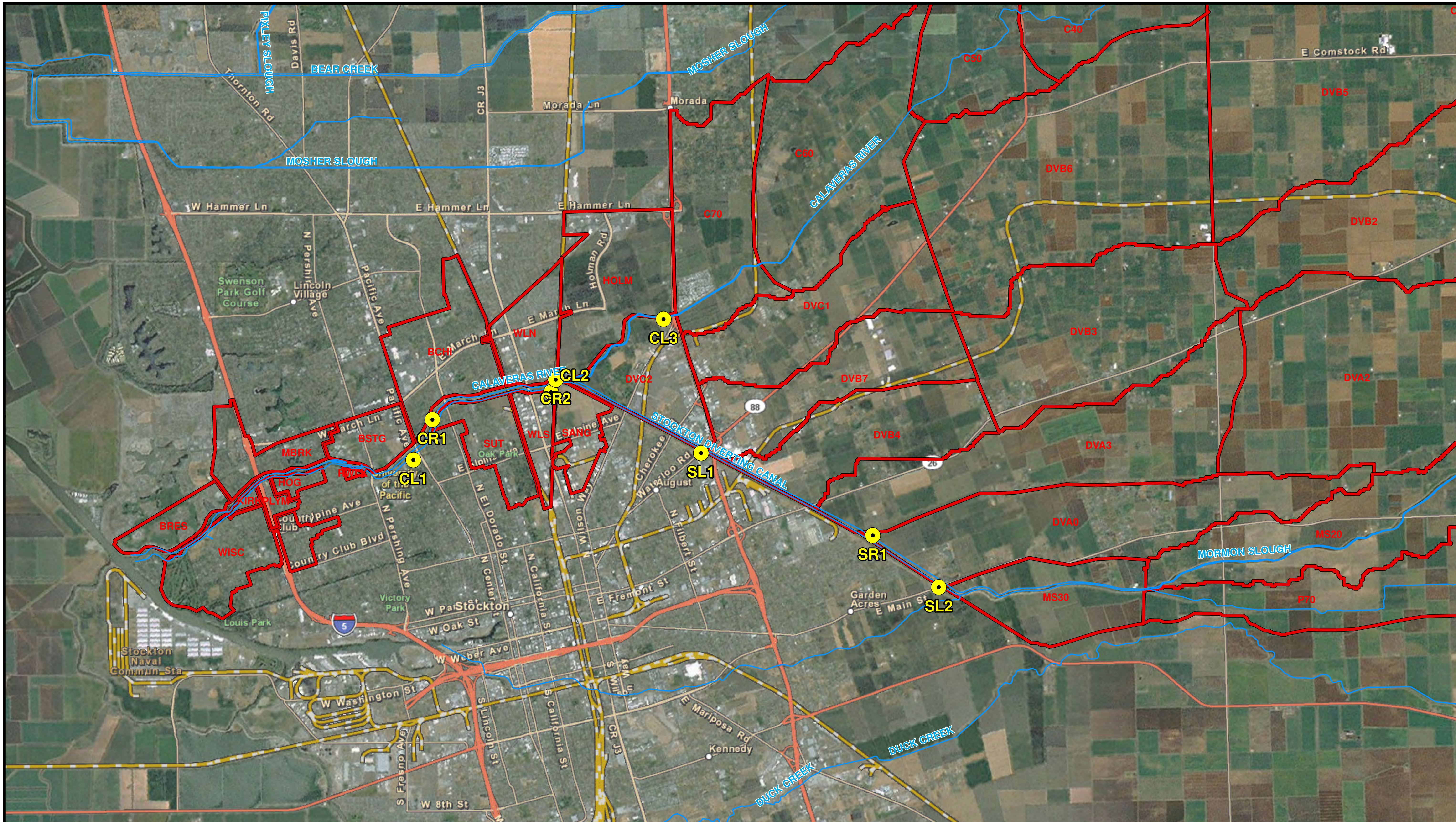
SAN JOAQUIN AREA FLOOD CONTROL AGENCY

CALAVERAS RIVER HEC-HMS SUBBASINS

FIGURE

5-2

Plate 33. Calaveras River HEC-HMS Subbasins



Subshed Boundary	LSJRFS Index Point

0 0.25 0.5 1 Miles

1 inch = 1 mile

JUNE 20, 2012

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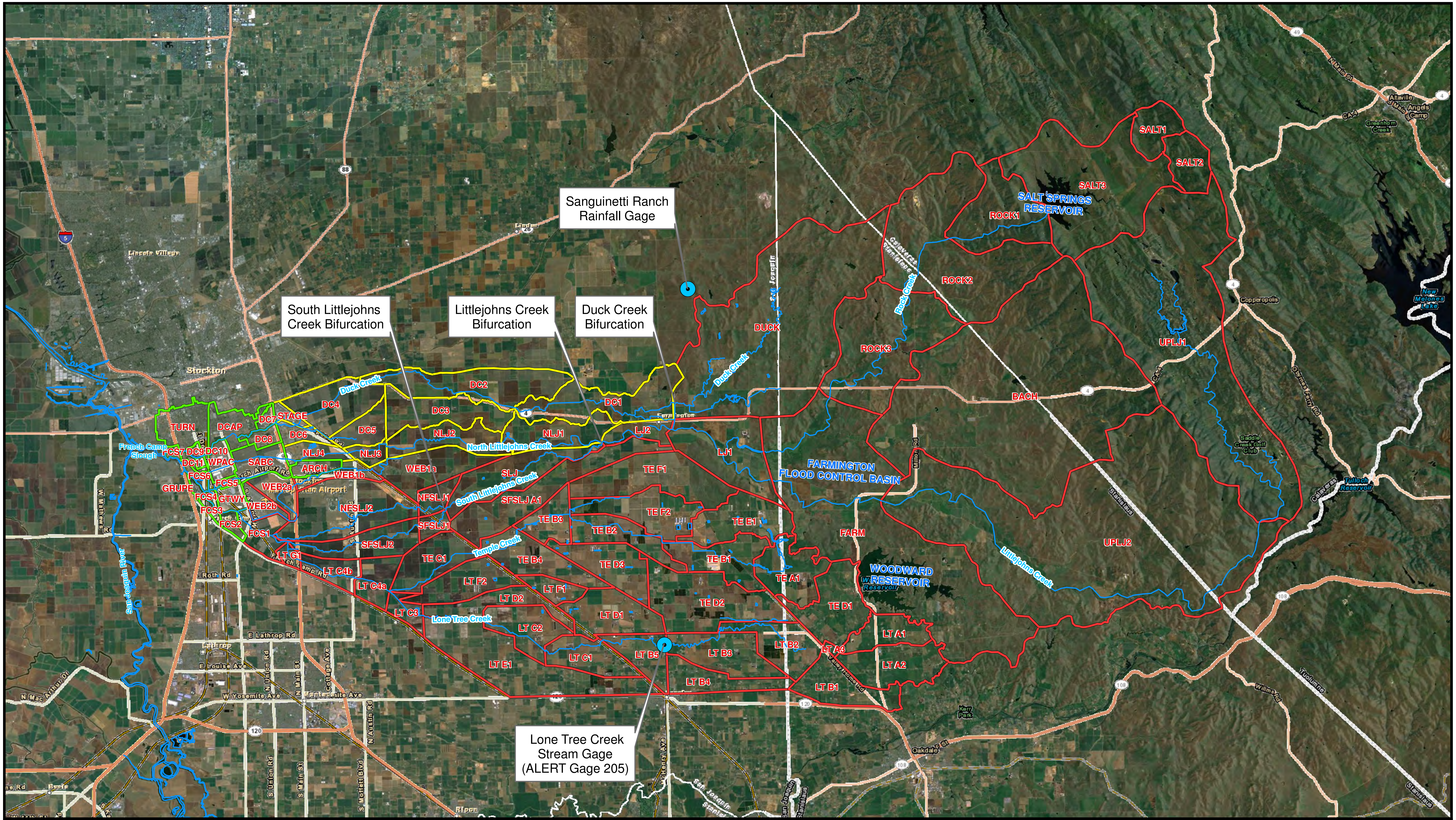
LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

CALAVERAS RIVER WATERSHED INDEX POINTS

FIGURE 5-12

Plate 34. Calaveras River Watershed Index Points



Subbasins from the Tidewater Model	
Subbasins from the Mariposa Lakes Model	
Subbasins Added by PBI	

N

0 1.5 3 Miles

1 inch = 3 miles

OCTOBER 19, 2010

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ENGINEERING . CONSULTING
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Folsom, CA 95630
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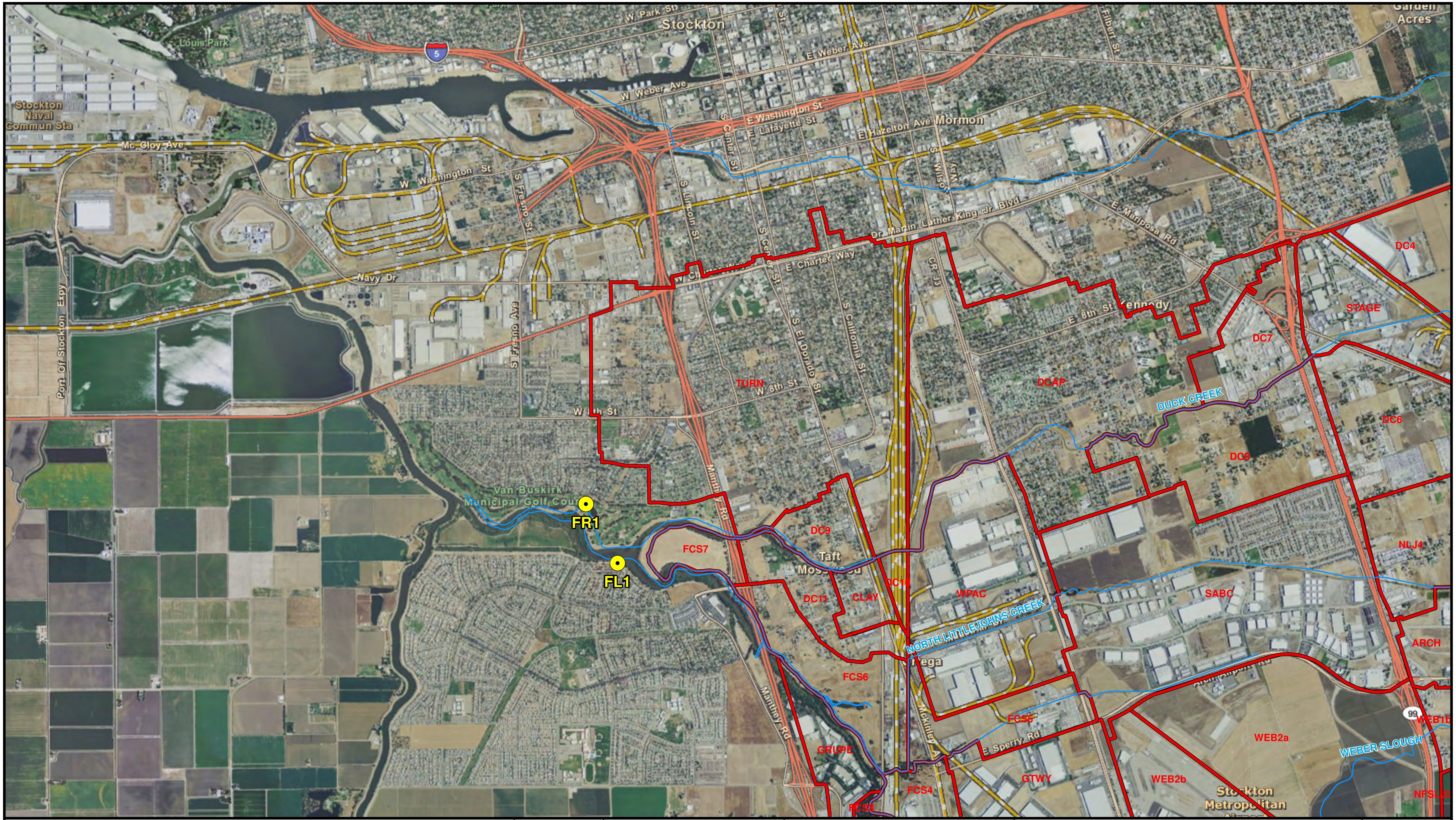
SAN JOAQUIN AREA FLOOD CONTROL AGENCY

FRENCH CAMP SLOUGH HEC-HMS SUBBASINS

FIGURE 6-2

Plate 35. French Camp Slough HEC-HMS Subbasins



Subshed Boundary

LSJRFS Index Point

0 0.125 0.25 0.5 Miles

1 inch = 1/2 mile

JUNE 20, 2012

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LOWER SAN JOAQUIN RIVER FEASIBILITY STUDY

FRENCH CAMP SLOUGH WATERSHED INDEX POINTS

FIGURE

6-11

Plate 36. French Camp Slough Watershed Index Points

Appendix 1

Lower San Joaquin River Feasibility Study
Calaveras River above Bellota
Hydrologic Analysis, 20 March 2014

US Army Corps
of Engineers.

Sacramento District

23 June 2014

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¶

Figures:

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Appendices:

1.0 Background

The Corps of Engineers, Sacramento District, Hydrology Section (SPK) tasked David Ford Consulting Engineers, Inc (DFC) with the derivation of unregulated and regulated flow-frequency curves at New Hogan Dam and Mormon Slough at Bellota (main control point for New Hogan Dam). Their report is titled: "Lower San Joaquin River feasibility study: Calaveras River frequency analysis and hydrographs" dated June 20, 2011. After DFC performed their analysis, revisions were made by SPK in February of 2012. These include 1) a newer version of HEC-ResSim was utilized for flood routing since the version DFC utilized had difficulty maintaining the objective flow release at Bellota – mainly due to local flow fluctuations 2) SPK reduced to four rather than nineteen the number of pattern floods used for scaling and routing through Res-Sim. As floods equal to or exceeding the 1% ACE event are the primary focus of alternatives in this study, SPK used only patterns that were representative of rare floods. The parts of the DFC analysis that remain valid and are incorporated into SPK's adopted hydrology are 1) unregulated frequency curve analyses including derivation of local flows during historic events 2) analysis of the critical duration and 3) the peak to volume characteristic curves. The parts of the DFC report that are superseded include their adopted unregulated to regulated transform and final regulated frequency curves at each index point. The DFC Report is attached to this Appendix and superseded sections have watermarks labeled "Superseded". The SPK report describes the final adopted hydrology for the feasibility study.

The lower watershed downstream of the Bellota gage was analyzed by Petersen Brustad, Inc (PBI) using a rainfall runoff model. See Chapter titled "Calaveras River Downstream of Bellota" for details on that analysis. The various frequency hydrographs developed at Bellota by SPK (as described in this chapter) became boundary condition input to the HMS model of the Calaveras River produced by PBI. One of the major purposes of the PBI model was to produce concurrent local flow hydrographs for areas downstream of Bellota, during a specific ACE flood event occurring at the Bellota gage. Levees are prevalent on lower Mormon Slough and the lower Calaveras River, which prevents local runoff from getting into the levees except by pumping. As such, a storm centered on the lower watershed will NOT produce the highest runoff within the levee system, needed for alternative analysis. A storm centered somewhere above the Bellota gage is important for modeling the levee system and economic damage areas.

It should be noted that an unregulated flow frequency curve at Bellota was the foundation for derivation of a regulated flow frequency curve at the Bellota gage. As such, the adopted regulated quantile flows are based on many different storm centerings that the gage has encountered during its long period of record.

The study area for the Calaveras River above Bellota is shown in figure 1 below.

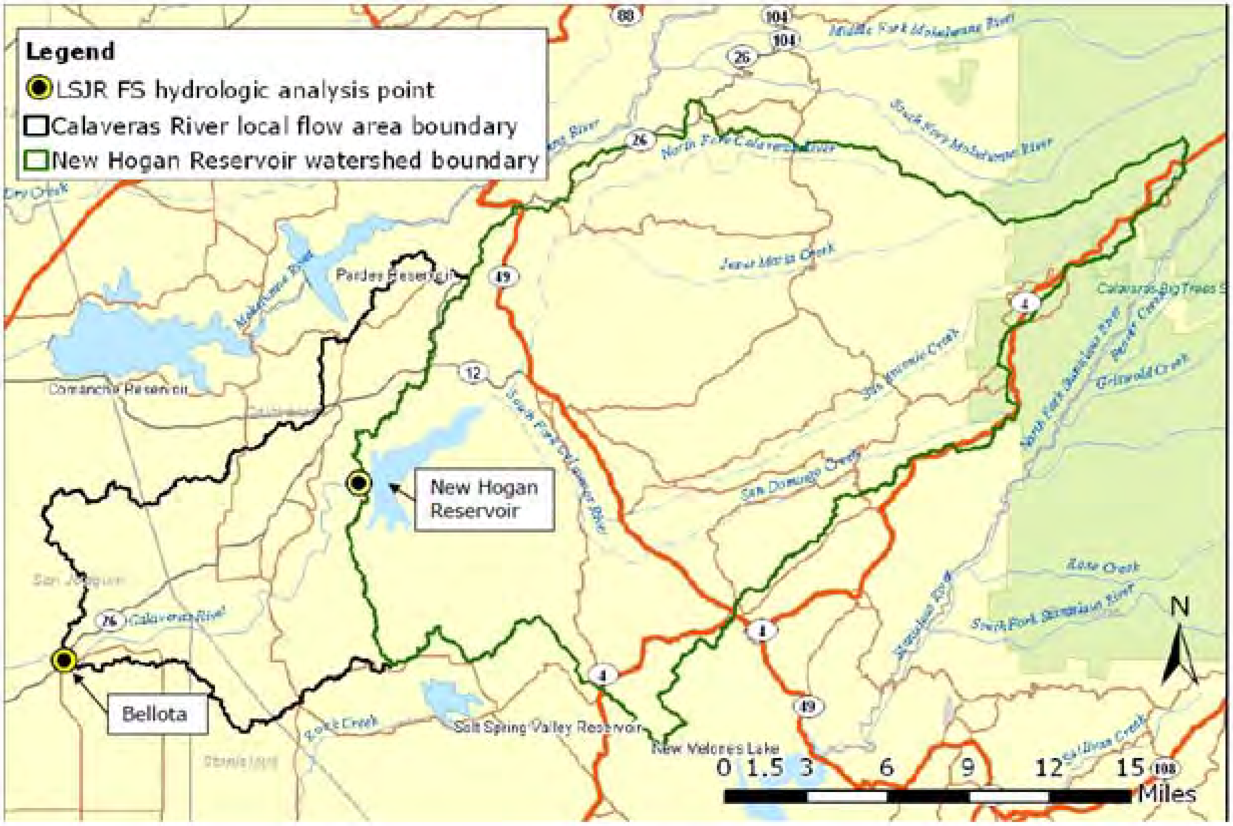


Figure 1. Calaveras River Study area

2.0 Watershed description

The watershed that is the subject of this report—the Calaveras River basin—is part of the lower San Joaquin River basin. It is located in Calaveras, San Joaquin, and Stanislaus counties. Located on Calaveras River approximately 28 miles upstream of Stockton, CA, is New Hogan Reservoir, a multipurpose facility with water supply, recreation, and flood control requirements. The Calaveras River basin encompasses 707 mi2. The north and south forks of the Calaveras River meet just east of New Hogan Reservoir and continue flowing into the reservoir. The basin comprises 3 major areas: The area above New Hogan Reservoir, which includes 363 square miles of relatively low-lying area on the western slopes of the Sierra Nevada. Elevations range from 550 ft at the dam to approximately 6,000 ft at the highest point. The 110 mi^2 area between New Hogan Reservoir and the downstream operation point at Bellota (the bifurcation of the Old Calaveras River and Mormon Slough approximately 18 miles downstream of the reservoir). The elevation at Bellota is approximately 130 feet. The remaining 234 mi^2 area of the Calaveras River and Mormon Slough watershed from Bellota to the confluence with the San Joaquin River. This portion of the watershed is low and flat with little topographic relief. Note: hydrological analysis of this region is completed by Petersen Brustad, Inc and is therefore beyond the scope of the analysis described here. The channel capacity downstream of New Hogan Reservoir is 12,500 cfs and the reservoir operates to limit flow to this value downstream of the dam and at Bellota (USACE 1983). A control structure exists at Bellota to divert the majority of flows into Mormon Slough. Downstream of this structure lies the Old Calaveras River channel, which is overgrown with vegetation. Flow is diverted into the Old Calaveras River when flow at Bellota reaches 13,500 cfs(USACE 1983).

3.0 Procedure for Analysis

The following steps were used to derive hydrographs for Mormon Slough at Bellota.

- Develop unregulated flow time series including New Hogan Dam inflow and local flow (between dam and the Bellota gage). This analysis was performed by DFC
- Develop 1-, 3-, 7-, 15-, and 30-day unregulated volume-frequency curves at New Hogan Reservoir and Mormon Slough at Bellota following the procedures in *Guidelines for determining flood flow frequency, Bulletin 17B* (IACWD 1982), EM 1110-2-1415 (USACE 1993) and using recent USGS regional skew analysis.
- If hourly unregulated flow is not available, convert daily unregulated hydrographs to hourly hydrographs using algorithm which preserves daily volume.
- Input historic and scaled unregulated hourly hydrographs into HEC-ResSim (both reservoir inflow and local flow) to create regulated hourly hydrographs at Bellota.
- Perform critical duration analysis at Bellota to determine volume duration that will be used in unregulated to regulated transform
- Fit at Bellota location, flow transforms to the event maxima datasets identified from the unregulated flow and corresponding simulated regulated time series.
- Developed a regulated flow-frequency curve and associated volumes by applying the flow transforms.
- Developed "expected" outflow hydrographs for Mormon Slough at Bellota for 8 flood frequencies: p=0.5, p=0.2, p=0.10, p=0.05, p=0.02, p=0.01, p=0.005 and p=0.002.   (Here the term expected hydrograph refers to a hydrograph that has a peak corresponding to the regulated flow frequency curve and associated volumes matching those from the family of characteristic curves corresponding to the given regulated peak flow.)

Figure 2 below illustrates the overall process.

The benefit of using multiple pattern floods events is that hydrograph shape, timing of runoff, and storm centering characteristics (spatial distribution of runoff) all result in different peak and volume runoff at index points. Modeling a hypothetical flood event using only one pattern does not account for the true variability of nature. Use of multiple patterns is more in line with USACE risk policies.

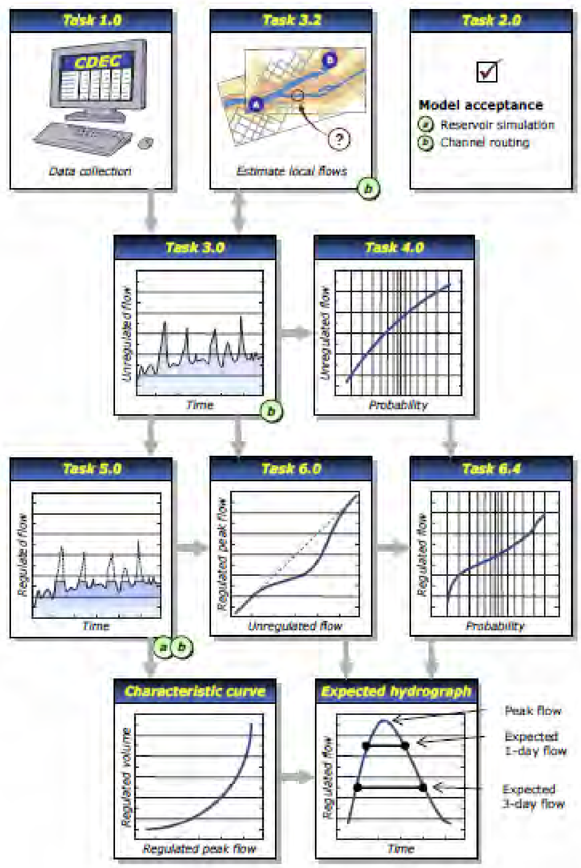


Figure 2: Process Flowchart

4.0 Unregulated flow time series development

SPK's Hydrology Section constructed unregulated flow time series at New Hogan Dam (for the Central Valley Hydrology Study) while DFC produced an unregulated times series at Mormon Slough at Bellota. DFC used the unregulated times series data provided by SPK for New Hogan Dam to construct the Bellota time series. DFC fitted unregulated volume-frequency curves for both of these locations. Thus, for unregulated conditions, the reservoir inflows were needed. For development of the unregulated flow time series downstream of the reservoir, a routing model was required to simulate the translation, attenuation, and combination of the unregulated flow hydrographs through the system. These flow hydrographs included the upstream boundary conditions (derived reservoir inflows) and intermediate area boundary conditions (estimated local flows). The routing yielded unregulated flow time series that served as the basis of: (1) the unregulated frequency analysis and (2) the unregulated-regulated flow transform. For this analysis, we developed an unregulated flow time series on the Calaveras River by: a) calculating daily unregulated reservoir inflow time series b) developing local flow time series for the area between New Hogan Reservoir and the reservoir's control point at Bellota d) completing the unregulated flow time series at the Bellota analysis point.

Obtain daily reservoir inflow. The Corps developed the daily unregulated reservoir inflow time series for New Hogan Reservoir using the continuity equation, in which, for a given time step, the average inflow equals the outflow plus the change in reservoir storage. For the calculation of these inflows, the source of the observed reservoir outflows and observed changes in storage was the Corps's database. By convention in the Central Valley, these calculations were completed on a 1-day time step, thus midnight to midnight values were used. This is consistent with the work completed for the *Sacramento and San Joaquin river basins comprehensive study* (Comp Study) completed in 2002 (USACE 2002).

Estimate local flow For the Calaveras River, local flows needed to be estimated for the area between New Hogan Reservoir and Bellota. Attachment 1 (page 52: Calaveras River local flow development) provides more details on this analysis. The estimation approaches used were:

• Option 1. Direct calculation of local flow using known releases from New Hogan Reservoir and the observed flows at Bellota, routing hourly flows as necessary. In the case of missing streamgage data, local flows values were interpolated as needed.

• Option 2. Estimation of local flows as:

$QLocal = 3.2(QCosgrove)$

where *QLocal* is the local flow estimate for a given time, and *QCosgrove* is the observed flow at the Cosgrove Creek near Valley Springs, CA, streamgage. The Corps estimates local flows for the purpose of real-time reservoir operations using this option (John High, personal communication, 11/9/2009).

• Option 3. Estimation local flows as:

$QLocal = 0.226\ (QNHG)$

where *QLocal* is the local flow estimate for a given time, and *QNHG* is the unregulated inflow to New Hogan Reservoir. The development of this relationship is show in Attachment 1

In Table 1 we summarize the selected approaches for local flow estimation on the Calaveras River by water year. This flow represents the total local flow contribution at Bellota. Attachment 1 provides details on the development of the local flow times series.

Table 1. Selected local flow estimation approaches for the area on the Calaveras River between New Hogan Reservoir and Bellota

Time period (water year) (1)	Time step (2)	Selected approach[1] (3)
1907-1929	Daily	Option 3: 0.226 times reservoir inflow.
1930-1969	Daily	Option 2: 3.2 times Cosgrove Creek flow.
1970-1987	Daily	Option 3: 0.226 times reservoir inflow.
1988	Daily	Option 1: directly calculate local flow.
1989	Daily	Option 3: 0.226 times reservoir inflow.
1990-1993	Daily	Option 1: directly calculate local flow.
1994-1995	Daily	Option 3: 0.226 times reservoir inflow.
1996-2009	Hourly	Option 1: directly calculate local flow.
2010	Daily	Option 2: 3.2 times Cosgrove Creek flow.

1. The approach listed is the predominant method for estimating local flows over the time period given. See Attachment 1 for further detail.

Complete unregulated flow time series

For the reservoir's operation point on the Calaveras River at Bellota, DFC combined the daily unregulated inflow time series with the estimated local flows by adding the 2 time series together. DFC did not route the unregulated reservoir inflows because: (1) synthesizing a shorter time step is not required for frequency analysis, and (2) the travel time between the reservoir and the operation point is approximately 7 hours, which is less than the 1-day time step of the inflows. In addition, there is little attenuation of flood peaks in this reach because of its length and channel geometry. DFC confirmed this by comparing observed releases from New Hogan Reservoir, observed flows on Cosgrove Creek, and observed flows on the Calaveras River at Bellota. Figure 3 displays the local flow area downstream of New Hogan Dam.

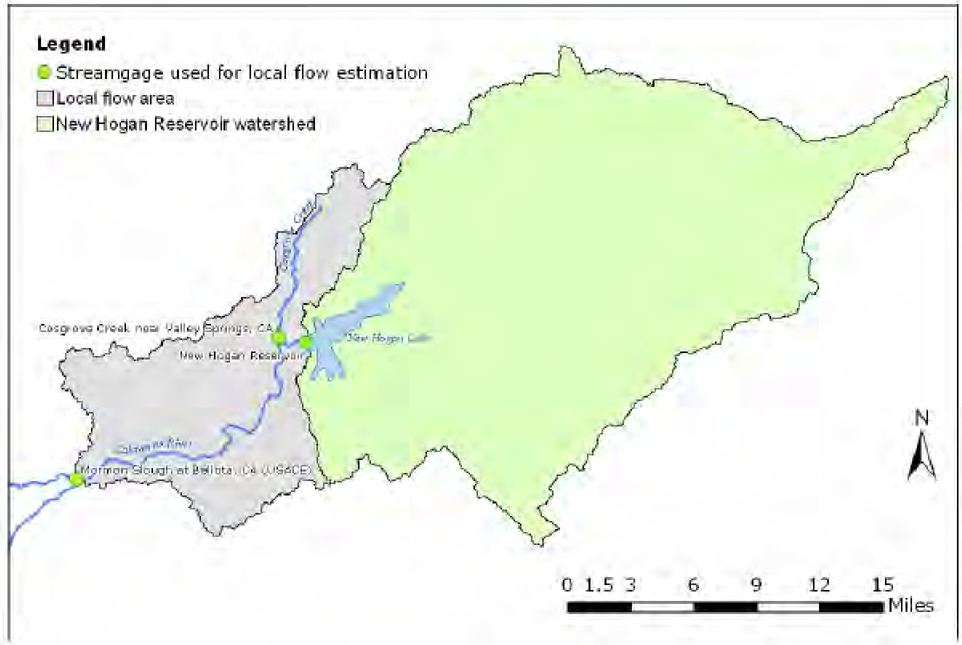


Figure 3: Local Flow Area Below New Hogan Dam

5.0 Unregulated frequency analysis

Accepted procedures to develop unregulated flow-frequency curves are specified in *Bulletin 17B* (IACWD 1982). The current standard-of-practice is to fit a Pearson III (LPIII) distribution to the logarithmic transforms of annual maximum series identified from streamgage data. Additional guidance for fitting frequency curves to volumes for a given duration is provided by EM 1110-2-1415 (USACE 1993). For this analysis, DFC used the unregulated inflows to New Hogan Reservoir to develop such an annual maximum series. However, because DFC only had records of regulated flows on the Calaveras River at Bellota, DFC could not fit a frequency curve directly using this method. Thus, DFC used the synthesized unregulated flow time series at this location and fitted a volume-frequency curve to that series. For this analysis DFC developed unregulated frequency curves that generally follow procedures specified in *Bulletin 17B* (IACWD 1982) with modification from the EMA procedure. This new procedure is being evaluated by the Bulletin 17C Committee for possible adoption for new federal guidelines for flow frequency. HQ USACE has given districts permission to use EMA. The EMA procedure includes different procedures for handling historic floods and a new outlier detection test called Multiple Grubbs-Beck. In some cases, the Multiple Grubbs-Beck test can result in a larger number of low outliers being censored than the Grubbs-Beck test used in Bulletin 17B.

For each analysis location, DFC:

• Identified the annual maximum series.

• (Task 4.1) Calculated regional skew values for each duration of interest using relationships developed by the USGS.

• (Task 4.2) Fitted LPIII distributions to the annual maximum series using the expected moment algorithm (EMA) enabled flow-frequency software PeakfqSA, version 0.937. This was

developed by Tim Cohn of the USGS and is based on the USGS's flow-frequency software PeakFQ (Cohn 2007).
• Reviewed and adopted the curves, checking them for consistency and comparing them to previously accepted values.

Identify annual maximum series
DFC identified the annual maximum series by extracting, from the unregulated flow time series, the volumes associated with the 1-, 3-, 7-, 15-, and 30-day durations. This information is detailed in attachment 1 (see pages 21 and 61). Note DFC developed a peak unregulated flow-frequency curve for New Hogan Reservoir for completeness; however this is not required for this analysis. In addition, DFC did not develop a peak flow-frequency curve for the Calaveras River at Bellota because the temporal resolution of the unregulated flow time series, 1 hour to as long as 1 day, is not an appropriate representation of instantaneous unregulated peak flow values.

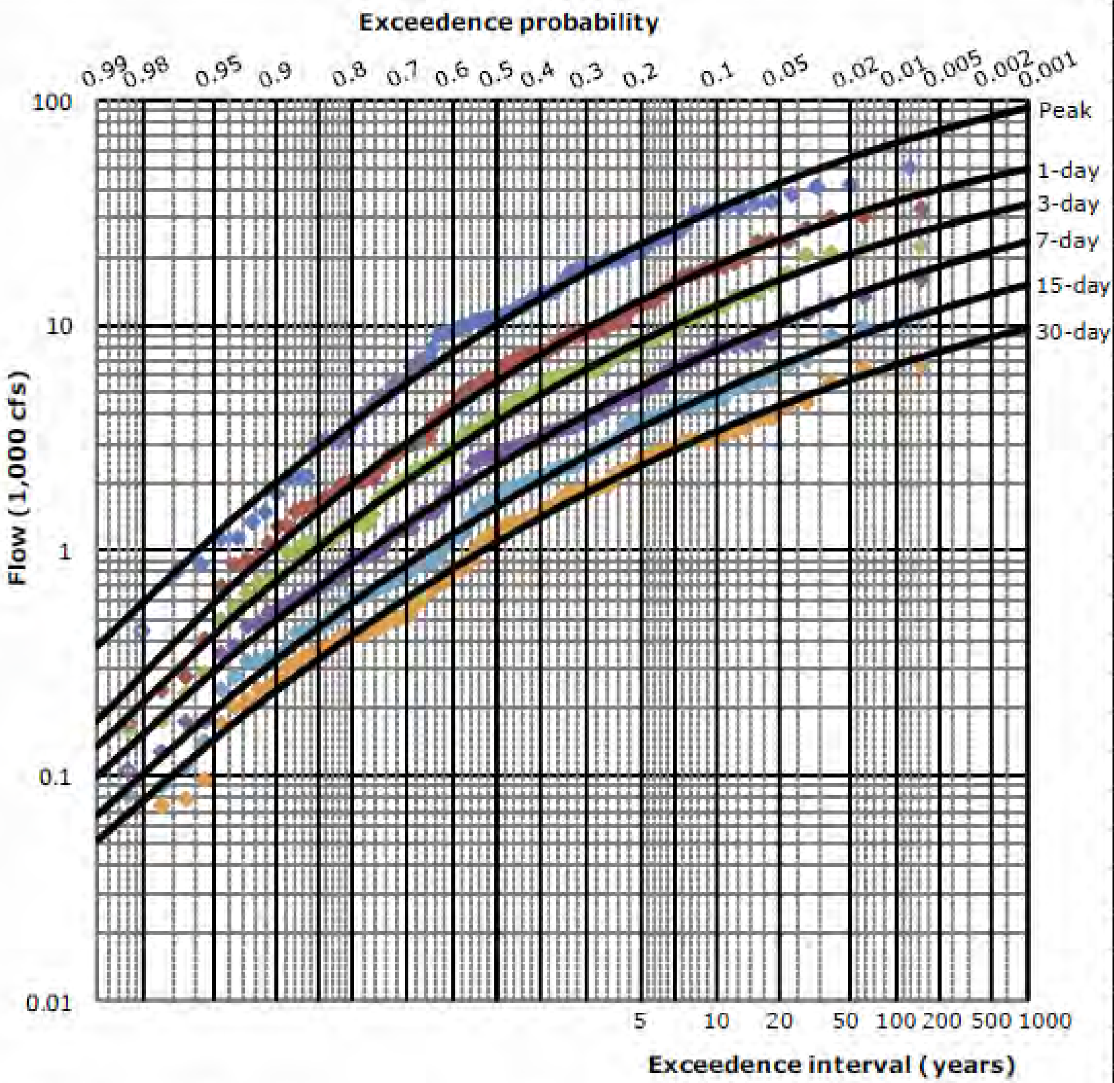
Calculate regional skew values
For this analysis, DFC calculated regional skew values for the peak flows and 1-, 3-, 7-, 15-, and 30-day volumes using the relationships developed by the USGS (USGS 2010). In these relationships, the regional skew value is a function of the average basin elevation. The values calculated for each analysis location and duration of interest are shown in attachment 1 (see page 76).

Fit frequency curves
To fit frequency curves to the annual maximum series DFC used: (1) the statistics of the logarithmic transforms of unregulated flow time series (mean, standard deviation, and skew), and (2) the regional skew values for the peak flow, and 1-, 3-, 7-, 15-, and 30-day calculated using relationships developed by the USGS (2010). The "at station" statistics were calculated using the EMA option in PeakfqSA. The weighted skew is automatically calculated by the PeakfqSA software used here.

Review and adopt curves
After fitting, DFC reviewed the frequency curves for consistency and appropriateness. Specifically, DFC:

• Compared the curve of a given duration to the curves associated with the other durations at the same analysis location.
• Compared the curves at a given location to the curves at the other analysis location to check for consistency.
• Compared the curves to those published in the Comp Study. DFC found the frequency curves on the Calaveras River were consistent between durations at each location. The curves do not "cross," and flow quantiles for a given duration at the downstream location are greater than those of the upstream location, as would be expected. As a comparison, DFC considered the volume-frequency curves developed for New Hogan Reservoir in the Comp Study (USACE 2002). The annual maximum series in the Comp Study ended in 1997. DFC also found that the flow quantiles of the curves fitted here and those of the Comp Study differ between the 2 sets of volume-duration curves by only 1% - 13%. The greatest differences (of 8%-13%) are in the 1-day volume quantiles. The 3-day and 7-day volume quantiles differ by only 1% to 5%. Peak

flow-frequency curves varied by as much as 9% because of the increased number of large events included in this analysis as compared to the Comp Study. DFC adopted the unregulated frequency curves for the two analysis locations, New Hogan Reservoir and Bellota, shown in Figure 4 and Figure 5. The detailed parameters used to fit these curves are included in Attachment 1 (see page 76).



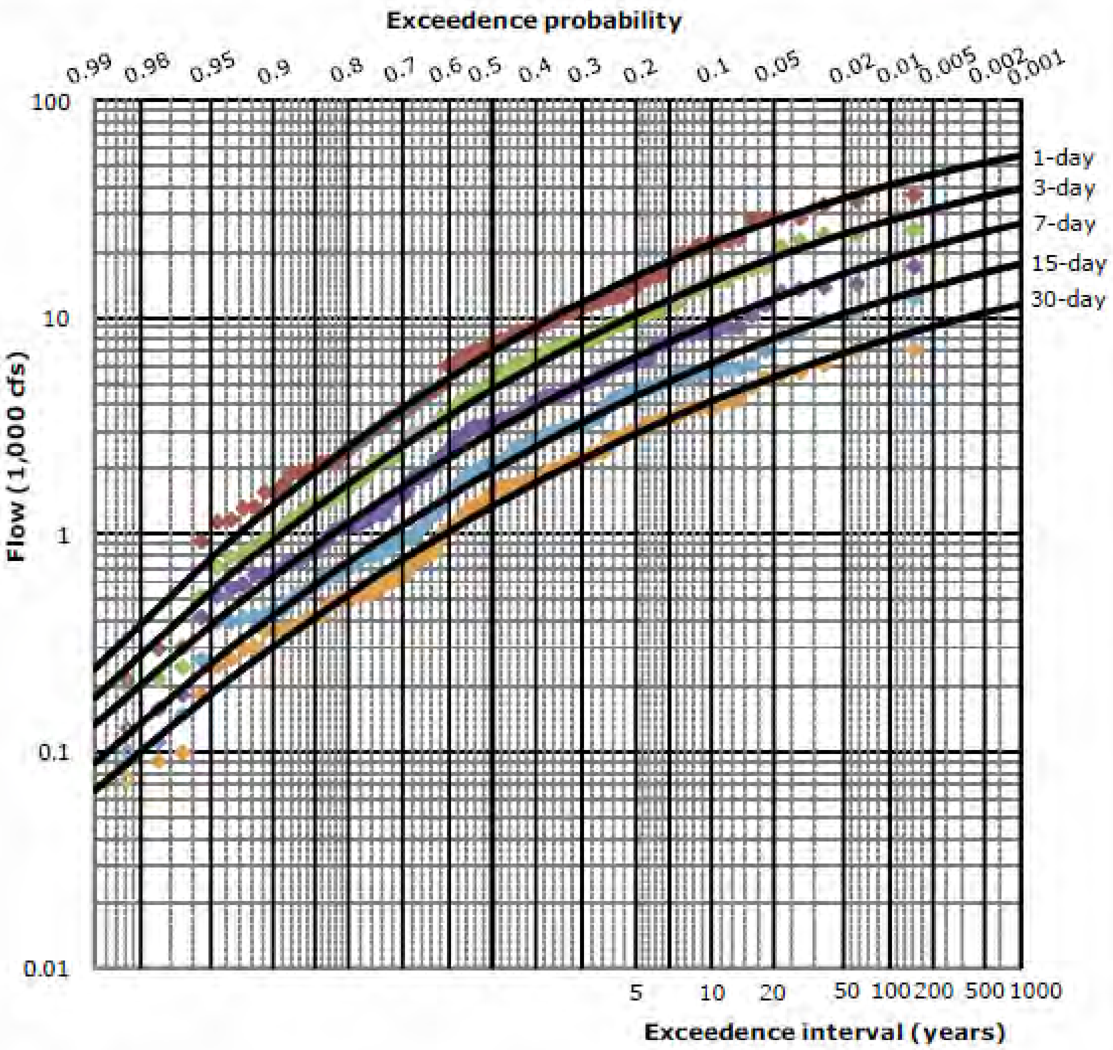
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.946	0.482	-0.727
1-day	3.685	0.501	-0.794
3-day	3.518	0.487	-0.732
7-day	3.324	0.477	-0.651
15-day	3.146	0.473	-0.656
30-day	2.988	0.457	-0.659

Notes:

- Median plotting positions.
- Drainage area: 363 sq. miles.
- Period of systematic record: 1907-2010.
  (Peak flow data intermittent 1930-2010).
- Record lengths
  Peak flows: 86 years.
  Volumes: 104 years.
- Regional skew values developed by USGS.
- Hollow points are censored events.

Figure 4: New Hogan Dam Unregulated Flow Frequency Curves
Note: Multiple Grubbs Beck test censored values shown as hollow points



Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
1-day	3.775	0.482	-0.810
3-day	3.608	0.475	-0.753
7-day	3.417	0.464	-0.666
15-day	3.240	0.461	-0.671
30-day	3.079	0.448	-0.668

Notes:

- Median plotting positions.
- Drainage area: 473 sq. miles.
- Period of systematic record: 1907-2010.
- Record length: 104 years.
- Regional skew values developed by USGS.
- Hollow points are censored events.

Figure 5: Mormon Slough at Bellota Unregulated Flow Frequency Curves
Note: Multiple Grubbs Beck test censored values shown as hollow points

Smooth unregulated flow time series The daily unregulated flow time series are appropriate for frequency analysis. However daily upstream and intermediate boundary conditions do not have the temporal resolution required by the CVHS procedures for assessing the effects of regulation, particularly releases as indicated on the emergency spillway release diagram (ESRD). Therefore, the daily reservoir inflows and daily estimated local flows were "smoothed" to hourly time series for input into HEC-ResSim by SPK staff. This smoothing was completed using a mass balance algorithm that interpolates the shape of the hydrograph and estimates peak hourly flows while maintaining daily volumes consistent with the original time series.

6.0 Regulated flow time series development

As mentioned before, SPK developed the adopted regulated times series for this study. To develop regulated flow-frequency curves, the unregulated volume duration- frequency curves are transformed through the unregulated- regulated flow transform. The unregulated-regulated flow transform captures the system's response to large, varied events, and is created using the unregulated and regulated flow time series data. To develop the regulated flow time series, SPK took four selected historical events (1956, 1936, 1938, and 1958) from the unregulated flow time series and simulated those in the regulated system using HEC-ResSim. In addition, SPK downscaled and upscaled the unregulated hourly pattern hydrographs and ran them through HEC-ResSim to represent a full range of different sized events. SPK then compiled the maximum unregulated and regulated flow data pairs for various durations to develop the event maxima datasets. These datasets became the basis for the unregulated to regulated transform development. To create transforms, one must first perform a critical duration analysis at each analysis point for the study.

Determine critical duration

DFC performed a critical duration analysis at two locations. Details on this analysis can be viewed in Attachment 1 (see page 81). In their analysis DFC identified the duration of the unregulated annual maximum series that consistently estimates the largest flow for each probability. In selecting the critical duration, they considered both the "goodness of fit" of each transform and which duration estimates the greater peak regulated flows. From their analysis, they determined that the critical duration at New Hogan Reservoir is 3.5 days, while at Bellota it is 1 day. Thus, the appropriate unregulated-regulated flow transforms used in this analysis were associated with these durations. The critical duration associated with the downstream operation point is shorter than that of the reservoir because of the effects of uncontrolled local flow. Local flow is not insignificant. A PBI rainfall runoff analysis with a calibrated model indicates that a 0.005 ACE storm centered between New Hogan Dam and the Bellota gage is capable of producing a peak flow of 12,500 cfs entirely from the local flow area (drainage area is approx. 100 square miles). 12,500 cfs is the objective flow at the Bellota control point in this watershed.

Table 2. Calaveras River floods-of-record at New Hogan Dam

Water year[1] (1)	Start date (2)	End date (3)	1-day max volume (cfs) (4)	Selection basis (5)
1958	3/10/1958	4/30/1958	32,920	*Large inflow event*
1938	1/25/1938	2/28/1938	30,450	*Large inflow event*
1911	1/10/1911	2/28/1911	30,175	Unreliable Local Flow
1936	2/10/1936	3/24/1936	26,987	*Large inflow event*
1907	3/1/1907	4/14/1907	23,641	Unreliable Local Flow
1986	2/10/1986	3/6/1986	23,494	Comp Study storm matrix event
1956	12/15/1955	2/15/1956	20,156	*Reasonable Local Flow Character*
1998	1/1/1998	3/15/1998	16,919	Comp Study storm matrix event
1997	12/1/1996	2/15/1997	16,801	Comp Study storm matrix event
1969[2]	1/5/1969	3/20/1969	14,674	Comp Study storm matrix event
1940	2/11/1940	3/16/1940	13,610	Comp Study storm matrix event
1965	12/18/1964	1/18/1965	12,789	Comp Study storm matrix event
1982	12/28/1981	1/31/1982	12,321	Comp Study storm matrix event
1983	2/25/1983	4/10/1983	10,433	Comp Study storm matrix event
1995	3/1/1995	4/6/1995	10,146	Comp Study storm matrix event
1951	11/12/1950	11/31/1950	9,390	Comp Study storm matrix event
1980	1/1/1980	1/31/1980	8,648	Comp Study storm matrix event
1967	1/20/1967	2/10/1967	6,738	Comp Study storm matrix event
1978	3/1/1978	3/19/1978	5,770	Comp Study storm matrix event

1. Events are in order of increasing 1-day flow volume
2. For the purposes of this analysis, treat the 1969 flood as 1 single event.
3. Pattern flood used for reservoir routing shown in italics font

Reservoir Regulation Simulation Criteria

SPK's Hydrology Section performed the final reservoir simulation in HEC-ResSim (version 3.1.8 RC4). This version corrected problems that DFC encountered when running an earlier version that was unable to keep the flow at Bellota to the objective channel flow of 12,500 cfs. At times, the older version of the model produced flows up to 14,000 cfs even though plenty of flood space remained behind the dam.

The HEC-ResSim model was developed as part of the Central Valley Hydrology Study. An Agency Technical Review (ATR) was performed by a retired annuitant working at HEC (Dan Barcellos). The model was setup to follow the rules in the latest approved Water Control Diagram.

Starting storage assumption: Starting storage is assumed to be bottom of flood control as defined in the Water Control Diagram. For each event modeled, 45 days of scaled historic inflow (including pre- and post-waves around the main flood wave) were ran for each simulation. One consistent ratio was applied to all ordinates of the historically based 45 day inflow hydrograph pattern. The purpose of the longer simulation was to partially compensate for the starting storage assumption, i.e. measure the impact of multiple waves of inflow to the dam over time upon its operation. Review of historic floods at New Hogan Dam indicate that starting at bottom of flood control is a reasonable assumption. Figure 6 shows the New Hogan Dam storage at the beginning of the 1997 flood event.

Adjustments for common floods: For the more common events, the antecedent storage condition might have the reservoir below bottom of flood control. In other words, there is water supply space available to absorb the inflow volume during an event. Another factor is that reservoir managers have a history of making releases at less than objective flow rates if forecasts indicate the event will be small. To compensate for these realities, SPK's Hydrology Section produced a graphical peak flow frequency curve at Bellota for the period after the dam was built. The gage record for this period includes both reservoir outflow and local flow. For probabilities of 0.5 to 0.04 ACE, the adopted regulated n-year hydrographs were adjusted to match the graphical peak curve based on historic data. Adjusting the hydrograph to match historic data for common events compensates for our starting storage assumptions, and for the decisions water managers make during these types of events.

Seasonal floods: The scaled events keep their historic time stamp in the dssfile when input into HEC-ResSim. The 1958 flood occurred in early April (maximum 1-day flow occurred April 3rd). The ResSim model has a smaller amount of flood space at this time of year due to the seasonality of the rule curve in the Water Control Diagram. As such, it turned out the 1958 flood pattern was the most difficult for the ResSim model to control. The probability assigned to the scaled 1958 floods came from the 1-day rainflood frequency curve which includes December through March flood events. This is a conservative way of estimating the probability of a specific flood occurring in spring. The true probability of such a flood occurring in April is best evaluated by performing a seasonal flow frequency analysis, which undoubtedly would assign it a more rare frequency than our current method. In hindsight, if SPK conducted this study a second time, it should take this into consideration. Since the median transform was used to define the adopted regulated frequency curve, the current use of the 1958 flood pattern did not adversely impact the outcome of the analysis since the 1958 transform fell on the high side of the four transforms produced.

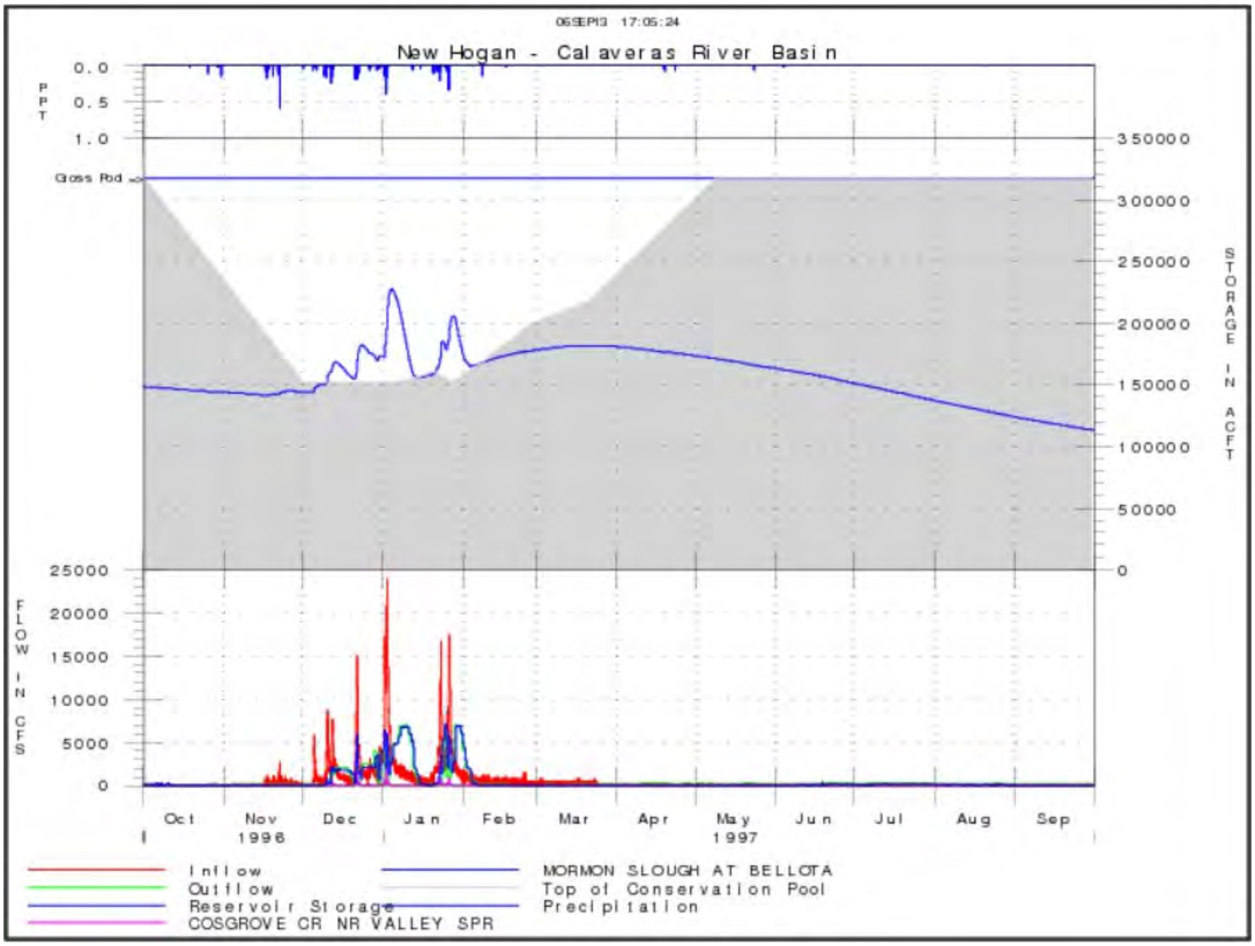


Figure 6: Storage at New Hogan Dam at start of 1997 flood event

Selection of Pattern Floods Used in ResSim Routings. The main focus of this feasibility study is to provide urban areas like Stockton flood protection from rare floods. Many tributaries studied in this feasibility study (such as Calaveras/Mormon Slough) currently have levees that were originally designed to provide protection from the 0.01 ACE event. The sponsors have a keen interest to achieve protection from the 0.005 ACE event. As such, SPK chose to pick some of the rarest historic events as a template for modeling alternatives in this watershed. The rarer flood patterns should also provide a better estimate of the local flow runoff that the reservoir will have to deal with when a really rare events occurs. Within the 104 years of recorded flow, the highest four ranking floods (ranked largest to smallest using the 1-day unregulated volume) are 1911, 1958, 1938, and 1936. 1911 was thrown out; however, because neither the Cosgrove Creek gage nor the Bellota gage were available to estimate local flow and therefore local flow had to be computed as a ratio of reservoir inflow (this method is considered the least accurate method of local flow estimation). The 1911 flood was replaced with the Dec 1955 flood because a) it was one of the most closely monitored/documented floods in the Central Valley and b) its local flow was within the range of variability of the other three large events used in this analysis (1958, 1938, and 1936). Table 3 below shows information about the selected patterns including local flow characteristics.

Event	Ranking by total 1-day unregulated volume	Hourly peak of total flow unregulated hydrograph (cfs)	Hourly peak of local flow hydrograph (cfs)	Percent local flow	Date of 1-Day maximum unregulated flow	Date of 1-day maximum local flow
1958	1	50,300	2190	4%	03 April	01 April
1938	2	46,400	3200	7%	11 Feb	11 Feb
1936	4	41,000	3800	9%	23 Feb	22 Feb
1956	7	30,300	2800	9%	23 Dec	23 Dec

Table 3: Selected Patterns for Res-Sim Routings

The choice of events was guided in part by the confidence in the local flow computations. The method of local flow computation by direct calculation of the difference between the historically observed hourly releases at New Hogan tailwater and the observed flow at Bellota is acceptable. Also acceptable is the method of local flow calculation by ratio of historically observed hourly flow at Bellota and at Cosgrove Creek at Valley Springs. The ratio of local flow at Bellota to the flow at Cosgrove Creek was found to be 3.2 by analysis of historic floods and is used for real-time water control decisions. The analysis was conducted by the District Hydrologist (Robert Collins) some years ago, although the details of the analysis are not currently available. The 1997 flood closely followed this rule as shown in Table 4. The computation of local flow by ratio with reservoir inflow is judged to be the least accurate as this relationship was found to be highly variable. Therefore, events where local flow was computed as a ratio of reservoir inflow were discarded for use in the regulated analyses. A comparison of the ratios of Bellota local flow to reservoir inflow and Cosgrove Creek flow for six historical events are shown in table 4 below.

Ratios of Bellota Local to New Hogan Inflow and Cosgrove Creek to Bellota Local for six flood events: 1965-1967-1986-1995-1997-1998. Copied from PORx1.0 simulation.dss by Ford.

Year of Event	Bellota Local	Bellota Frequency	New Hogan Reservoir Inflow	NewHogan Inflow Frequency	Cosgrove Creek	Cosgrove Frequency	Bellota Local / Res Inflow	Bellota Local / Cosgrove Creek
1965	2303.3	0.68	19000.0	0.25	N/A	N/A	12.1%	N/A
1969	1592.4	0.16	21900.0	0.15	N/A	N/A	7.3%	N/A
1986	5849.5	0.11	35500.0	0.04	N/A	N/A	16.5%	N/A
1995	2720.8	0.65	14900.0	0.39	N/A	N/A	18.3%	N/A
1997	6688.3	0.12	25100.0	0.17	2048.0	0.60	26.6%	326.6%
1998	9436.0	0.04	25300.0	0.20	2396.0	0.18	37.3%	393.8%
Average ratio from report; Value * ratio = Bellota Local =>				22.6%	320.0%			

Table 4: Ratios of Bellota Local Flow to New Hogan Dam Inflow or Cosgrove Creek

In summary, since rare floods like the 0.005 ACE event is important for the evaluation of alternatives in this feasibility study, the rarest events were selected as pattern floods to scale and route through HEC-ResSim. The local flow that occurred during these large events is considered the best representation of what might happen in a flood of this magnitude. The 1911 event was thrown out because there is not confidence in the method needed to estimate local flow for this event (Option 3 – ratio of reservoir inflow).

Validating the Transform: USACE guidance indicates that a local flow frequency curve should be developed to determine the lower boundary of a regulated frequency curve developed from an unregulated to regulated transform based on reservoir routings. Theoretically, the transform can exceed the local flow frequency curve but should not fall below it. This is due to the fact that the local flow cannot be controlled and therefore will always impact an analysis point. Local flow does not include reservoir releases. Two estimates of local flow runoff were attempted.

First attempt: DFC derived a "Limited Use Frequency Curve" for peak and 1-day durations using 14 and 19 years of record, respectively. This was the number of water years in which the Option 1 method of local flow calculation was available. Figure 7 below displays the curves. DFC termed it as "Limited Use" because a) it does not include reservoir releases and 2) it was based on a limited number of years of data. The DFC "Limited Use Curve" is provided in this report for interest only and was not utilized in this study, other than to help verify the transform at Bellota was reasonable. The maximums derived for these two curves do not necessarily represent annual maximums, although typically maximum local flow does occur approximately the same time (within a few days) whenever New Hogan Dam has the largest inflow of the water year. Instead, the data used represents the peak local flow runoff that occurred within the 45 day window of the selected flood event that DFC analyzed for each water year where local flow could be calculated using Method 1. Table 5 displays the various quantiles computed from this curve. The adopted transform at Bellota *does not* fall below the Limited Use Frequency Curve for all frequencies (except the 0.005 ACE event). Since a flow frequency curve based on 14 years of data is highly suspect at the upper end due to the small sample size, the curve was not really used for the study. As mentioned later in this report, the 0.50 to 0.04 ACE event hydrographs were modified to match a family of graphical flow frequency curve at Bellota (these curves include both local flow and reservoir releases). For rarer floods, SPK decided to use the PBI calibrated rainfall runoff model with a storm centering above Bellota to estimate local runoff potential for floods equal to or rarer than the 0.02 ACE event. Again, DFC's Limited Use Frequency Curve is presented here for interest only but the study results did not depend on it.

DFC performed a coincidence analysis to determine the relationship of New Hogan Dam inflow and local flow at Bellota (page 25 of attached DFC Report). This was done out of concern that scaling dam inflow and local flow by the same factors may result in local flow that becomes too rare. Figure 19 of DFC Report shows the probability of local versus New Hogan inflow for selected flood patterns and scalings. Ufortunately, the frequency of local flow is appraised with DFC's "Limited Use" flow frequency (14 years of data) which is not very trustworthy. As such, the results are ***inconclusive.***The plot appears to show that 1) local flow is highly variable depending upon the flood event and 2) scaling local flow hydrographs (see values for same color pattern) might not significantly change relationship between reservoir inflow and local flow.

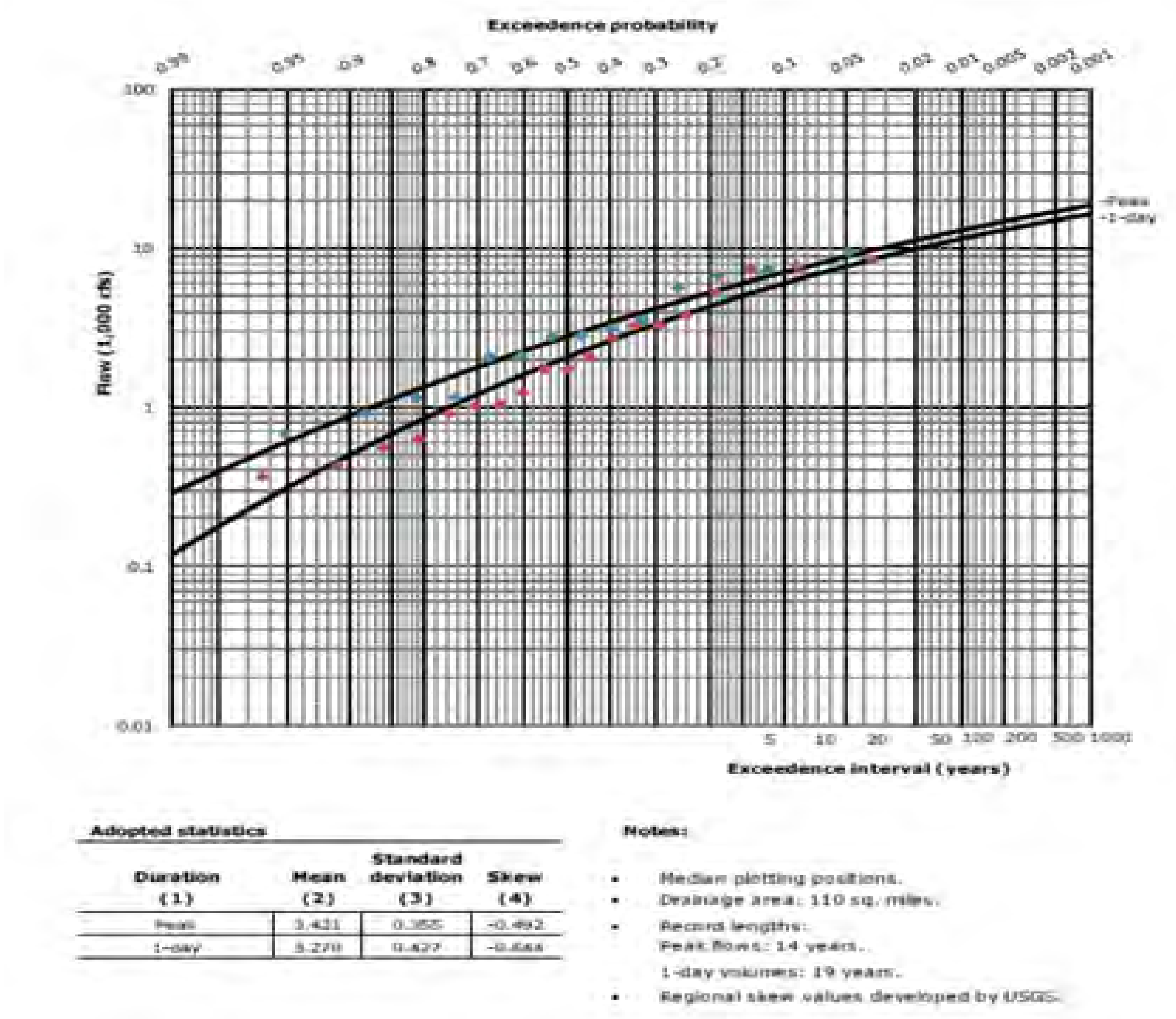


Figure 7: Limited Use "Local Flow" frequency curve (not used in study).

Annual exceedence probability (1)	1/annual exceedence probability (2)	Peak flow (cfs) (3)	1-day volume (cfs) (4)
0.500	2	2,817	2,067
0.200	5	5,310	4,324
0.100	10	7,134	6,015
0.050	20	8,942	7,688
0.020	50	11,318	9,855
0.010	100	13,103	11,449
0.005	200	14,874	12,995
0.002	500	17,188	14,957

Table 5: *Limited Use "Local Flow" Frequency Curve for Mormon Slough at Bellota

*Note: This curve was not used in this study. Presented for interest only. Does not include New Hogan Dam releases. Based on 14 and 19 yrs of data for the peak and 1-day durations.

2nd Attempt: For the overall study, PBI developed a calibrated rainfall runoff model for the lower watershed below New Hogan Dam. The study results of their analysis are discussed in Chapter D (Calaveras River Downstream of Bellota). The model was calibrated to the Bellota gage for a historic storm. After building a calibrated model, an attempt was made to estimate the local flow runoff potential including for the 0.005% ACE event. PBI input two different 0.005 ACE design storms into their calibrated model that were centered between the dam and the Bellota gage. One design storm was the hypothetical, pyramid shaped, storm within HMS that was fully balanced to multiple-duration depths found in NOAA14 and using TP40 areal reduction factors (these factors are built into HMS). The other storm used a 72-hour, 1997 hyetograph pattern that was balanced to only the 72-hour, 0.005 ACE NOAA14 depth and using the HMR 59 areal reduction factor for this duration. In both cases, the resulting peak flow at Bellota in their model was 12,500 cfs. PBI also input various frequency storms centered between Bellota and the dam to get a handle on local flow frequency. The results of those runs is shown in Table 6 below. Except for the 0.50 (2-year event), the transform at Bellota (transform based on the reservoir modeling of both reservoir outflow and local flow combined) did not fall below the local flow runoff peak predicted by PBI's model. Since peak flow frequency at this location was adopted from the graphical regulated frequency curve at Bellota based on 23 years of data, the transform was not used for any events more common than the 0.02 ACE. The PBI analysis results helped validate SPK's transform was reasonable for events more rare than the 0.04 ACE event. This is further explained below.

Peak Runoff- Local Flows at Bellota [cfs]			
	Storm Centering		
Storm Frequency	Urban	Bellota	Above New Hogan
2-year	3,270	4,600	2,660
5-year	4,430	6,190	3,620
10-year	5,400	7,430	4,390
25-year	6,640	9,020	5,470
50-year	7,570	10,210	6,270
100-year	8,470	11,380	7,060
200-year	9,370	12,540	7,830
500-year	11,040	14,430	9,240

Table 6: Bellota local flow peaks for storm centerings by PBI.

Note: The storm centered between New Hogan Dam and Bellota (3rd column labeled "Bellota") produced the highest local flow runoff.

0.005 ACE Event: The results of the ResSim modeling (specifically the adopted regulated flow frequency curve) indicate the 0.005 ACE runoff for the Mormon Slough at Bellota analysis point is 12,500 cfs. This may seem to contradict the fact that the local flow runoff is also estimated to be 12,500 cfs for the same frequency event based on rainfall runoff modeling. The discrepancy can be explained as follows:

a) As Table 4 above indicates, the relationship between New Hogan Dam inflow and local flow runoff is highly variable and not well correlated. The possibility of a 0.005 ACE release from New Hogan Dam and a 0.005 ACE local flow runoff during the same flood event is considered highly unlikely based on Table 4. In fact, for the three largest floods in which local flow can be reasonably calculated (1958, 1936, and 1938), the local flow peak never exceeded 4,000 cfs. 4,000 cfs is approximately a 0.20 ACE (5-year return period) flood based on the DFC Limited Use frequency curve, which implies that the two watershed areas (above and below the dam) are not highly correlated during extreme storms. Another factor is that the maximum local flow runoff sometimes occurs earlier than the peak of the reservoir inflow hydrograph. See the last column of Table 3.

b) The New Hogan Dam Water Control Manual specifically requires the dam to keep releases to no more than 12,500 cfs at Bellota. The rules force the dam to cut back on releases if local flow is high. A separate analysis by DFC at New Hogan Dam indicated the reservoir could keep its releases to about 12,500 cfs (just downstream of the dam) during a 0.5% ACE inflow event if the dam does not have to adjust for downstream local flow. See Attachment 2. Historically, the local flow runoff tends to peak about the same time or earlier than the peak of the reservoir inflow hydrograph. Since the reservoir can delay its maximum releases beyond the time of its maximum inflow, the local flow has a chance to pass downstream before large releases from the dam are necessary (in other words timing comes into play). The above stated facts help explain why the flow at Bellota can be maintained at 12,500 cfs during this size event for some patterns in SPK's ResSim model.

8.0 Create Mormon Slough at Bellota Hydrographs for Specific Frequencies

The following steps were performed to extract an outflow hydrograph for each “n-year” event corresponding to the regulated flow-frequency curve for Mormon Slough at Bellota.

1. Simulate the 1936, 1938, 1956, and 1958 events with HEC-ResSim version 3.1.8 RC4. This version corrects defects in the downstream rule logic. These simulations correspond to the development of regulated flow time series in the DFC report. These simulations develop regulated flow time series for scale factors from 1.0 to 3.0 of reservoir inflow and local flow, which are input to the simulation model. The four events were chosen out of a list of the highest floods of record.

2. Extract the 1-day unregulated flow volume and regulated peak flow at Bellota from the DSS files output from simulations in step 1. The 1-day unregulated flow volume was identified as the “critical duration” by DFC in Attachment 1 (see page 81) for the .02 to 0.005 ACE events. So, the independent variable (x-axis) of the flow-flow transform is the 1-day unregulated flow, with the peak regulated flow being the dependent (y-axis) value. Then use a spreadsheet to input the 1-day unregulated flow and peak regulated flow data pairs to compute the transform for each pattern. SPK’s Hydrology Section decided to adopt the median transform to develop a regulated peak flow frequency curve. To compute the median curve, an average regulated peak flow value (y-axis) is computed for each x value from the two innermost transforms (note: we developed four transforms). Figure 8 displays the four individual event based transforms plus the average and median transforms for the Bellota gage location. Table 7 displays individual values from the average and median transforms. The median transform was adopted for the study.

3. The regulated hydrographs for the 0.5 to 0.04 ACE flows at Mormon Slough at Bellota were *revised to fit observed conditions at the Bellota gage* via a family of graphical curves using 23 years of historic data (water years 1988 to 2010). It is noted that using this approach may limit the ability of the District to evaluate alternatives involving reservoir reoperation or reconfiguration. This is because it is not possible to generate equivalent graphical frequency curves for with-project conditions. Currently, reservoir reoperation is not one of the alternatives being moved forward in the analysis. The methodology described above uses the HEC-ResSim program, with unimpaired inflow data input to the reservoir and local flow areas, with operational rules documented in the Water Control Manuals. This provides a consistent reservoir operation that follows the Congressionally authorized plan of operation. In actual operation as shown by the historically observed flows, the reservoir was operated differently. That is, for smaller, frequent events, the reservoir was not drawn down as quickly as the water control plan suggests, but holds runoff in storage longer while making smaller, lower, releases. For example, during the 1997 flood event, the peak of the simulated release from the dam using HEC-ResSim was 12,500 cfs while the historic release was only 7,500 cfs. Figure 9 shows the actual operation for the January 1997 flood, while Figure 10 shows the hypothetical operations (note: the inflow hydrograph for the hypothetical simulation is derived from daily inflow values smoothed into hourly values using an algorithm which preserves the historic daily volume). Besides modifying the peak of the hydrograph for these frequency events, the volume was also modified to match a frequency analysis of historically observed flows. The runoff volume was found by computing the 1, 3, 7, and 15-day flow

volumes from historic daily regulated flow time series at Bellota, and then extracting annual maximums and computing the plotting positions of the resulting annual maximums, then interpolating the 0.5 to 0.04 ACE flow magnitudes. The derived values are shown in Table 8 below. The following steps were taken to produce hydrographs for these frequencies:

a. For the target frequency, select a 1997 pattern hydrograph with the scale factor that provides the proper unregulated volume based on critical duration (1-day for Bellota) unregulated frequency curve.
b. Based on the scale factor chosen in (a) above, obtain the corresponding Res-Sim output hydrograph at Bellota.
c. For the target frequency, find the appropriate peak flow and volumes from the graphical regulated frequency curves (Table 8).
d. Input the regulated hydrograph found in step b and the peak and volumes found in step c into HyBART in order to balance/adjust the hydrograph.

4. For the 0.02 to 0.002 ACE events, regulated peak flows were derived by the unregulated to regulated transform method in Figure 8. The procedure to derive final regulated hydrographs is described below.

   a. For the target frequency, select a 1997 pattern hydrograph with the scale factor that provides the proper unregulated volume based on critical duration (1-day for Bellota) unregulated frequency curve.
   b. Based on the scale factor chosen in (a) above, obtain the corresponding Res-Sim output hydrograph at Bellota.
   c. For the target frequency, find the appropriate peak flow (from the transform in Figure 8) and the concurrent volumes based on the DFC peak to volume regression analyses. DFC analyzed regulated peak flow to volume relationships from a regression analysis using multiple pattern events. The analysis was based on routing scaled historic flood patterns through Res-Sim and analyzing the resulting regulated flow hydrographs to obtain matching peak and volume data pairs. The data pairs were then used in a regression analyses, with peak being the known value x and volume being the prediction value y. Relationships were derived by DFC for regulated peak to regulated 1-, 3-, 7-, 15-, and 30-day volumes. The DFC analysis can be viewed in attachment 1 (see page 89).
   d. Input the regulated hydrograph found in step b and the peak and volumes found in step c into HyBART in order to balance/adjust the hydrograph.
   e. Create plot similar to the one shown in Figure 11 based on all hydrographs produced in HyBART including the 0.5 to 0.04 ACE events. Perform additional smoothing on the hydrograph volumes in HyBART for the 0.02 and 0.01 ACE frequency hydrographs to facilitate consistency between all frequencies so that the lines do not cross each other. The final adopted peak and volumes are plotted in Figure 11. Note: The 0.5 to 0.04 frequency hydrographs remain consistent with the family of graphical curves base on 23 years of data while the 0.005 and 0.002 ACE event hydrographs generally follow the DFC peak to volume relationships. Smoothing was performed on the 0.02 and 0.01 ACE hydrographs to achieve consistency in the plot in Figure 11.

In summary, Table 9 displays the final adopted regulated peak and volumes for each frequency event. Table 9 values were input to the program HyBART, a hydrograph balancing routine, along with pattern hydrographs from Res-Sim simulations of the 1997 flood. Simulated patterns were used rather than the actual observed pattern as the simulated and observed patterns are significantly different. The program HyBART creates balanced hydrographs that match the regulated peak flows in table 9 and follow the pattern of the 1997 flood event. HyBART creates a balanced hydrograph using all input peak flows and volumes. The Res-Sim model output hydrograph most closely associated with a specific frequency was selected as the input hydrograph for HyBART prior to balancing. For interest, the 1997 flood event pattern hydrographs for scale factors of the observed flood from 1.0 to 2.6 are shown in figure 12.

The resulting regulated flow hydrographs for the 0.5 annual chance exceedance probability (ACE) to 0.002 ACE events are consolidated in the spreadsheet: MSB-RegFlowFreq-1997SimPattern-Hydrographs.xlsx. A plot of the balanced regulated flows are shown below in figure 13. The hydrographs in figure 13 were eventually provided to PBI to route through their HEC-HMS model to compute additional hydrographs for index points downstream of Bellota in the Calaveras River watershed. The PBI model used a 1997 pattern storm to compute concurrent local runoff from sub-basins located downstream of the Bellota gage.

The DFC Limited Use flow-frequency curve was developed as a best fit analytical frequency curve of a 14 year period of historic data developed by subtracting lagged reservoir releases from observed flows at Bellota (reflective of local flow frequency only); whereas the flow-frequency for the 0.5 to 0.04 ACE events in table 8 were adopted from a graphical frequency curve based on a 23 year period of observed regulated flow (including local flow and reservoir releases at Bellota) after New Hogan dam was built. As only 23 years of record are available, the graphical curve is only useful for predicting peak and volumes for events *no rarer than* the 0.04 ACE (25-year return period). Although this is an apple to orange comparison, the values between the two frequency curves are substantially different only at the 0.5 ACE (2-year) frequency.

The 1997 event was chosen as the one event for producing specific frequency floods for the following reasons: a) It was a recent event in which hourly **hyetograph patterns** were available b) The various frequency hydrographs produced in this analysis became input to the HMS model produced by PBI, wherein the PBI rainfall runoff model produced concurrent runoff for areas downstream of the Bellota gage. c) In order to synchronize the two efforts, the same flood event (1997 flood) needed to be modeled in order for the timing of the total watershed runoff to be consistent with a real event.

9.0 Risk Parameter for the FDA Program

USACE policy is to use risk analysis as part of its planning and design processes. SPK's Hydrology Section is assigned the task of providing hydrologic risk parameters for use in the Flood Damage Analysis (FDA) program. The assignment of a period of record for the flow frequency curve input into FDA for each study index point is important as it defines the confidence limits about the curve. Here are some guiding thoughts on that parameter for the lower Calaveras River watershed. The assigned period of record for Mormon Slough at Bellota and index points downstream (Mormon Slough and Calaveras River) is 52 years. The critical duration for Mormon Slough at Bellota was determined to be 1-day. As the runoff at Bellota is a combination of both reservoir releases (driven by volume of inflow into the dam) and local flow, using a volume duration curve (as opposed to a peak curve) is acceptable. The 1-day unregulated flow frequency curve at Bellota has a 104 year period of record. Factors for this decision are as follows:

The HEC-ResSim model ResSim version 3.1.8 RC4 used in this hydrologic analysis is quite adept at figuring out how to adjust reservoir releases to maintain downstream channel capacity while accounting for the rise and fall of the local flow hydrograph at the Bellota gage. This is due to 1) the reservoir release logic imbedded in HEC-ResSim is quite complex and iterative 2) the model is given perfect foresight into the future to see the local flow hydrograph. For these reasons, the model may be too efficient in using the full downstream channel capacity; whereas a human operator would be more cautious without the perfect foresight. Currently, the Water Management Section of SPK uses the real-time gage on Cosgrove Creek to predict local flow (Cosgrove Creek x 3.2 = total local flow at Bellota). This relationship was determined by the District Hydrologist working at SPK and was based on evaluation of historic data. Prior to real-time data being available at Cosgrove Creek, the regulated flow at Bellota did exceed 12,500 by more than a thousand cfs when the New Hogan Dam water managers miscalculated the local flow runoff during the 1986 flood. The Cosgrove Creek daily recording device was re-established in 1991 after a long period of being unavailable. While the availability of real-time Cosgrove Creek flow measurements aids in the local flow estimation, a human operator may still be reticent to assume that the “Cosgrove Creek measured flow times 3.2 = total local flow at Bellota” rule is infallible. As such a human operator would probably release less than the reservoir model, which would have the impact of filling up the reservoir storage faster. Under these circumstances, the reservoir would provide a lower level of protection from extremely rare floods since the downstream channel is being used less efficiently.

Another factor in this discussion is the method in which both reservoir inflow and local flow are scaled by the same factor for routing through the HEC-ResSim model. From experience with the Central Valley Hydrology Study, SPK has learned that scaling reservoir inflow and local flow by the same factor can sometimes result in a conservative estimate of local flow. The standard deviation and skew of reservoir inflow frequency curve and the local flow frequency curve are often quite different. Typically, the local flow frequency curve flattens out at the upper end while the reservoir inflow frequency curve keeps rising (higher standard deviation). This is because the upper watershed above the reservoir has higher rainfall depths in the mountains due to orographic effects, which results in a higher standard deviation (steeper slope of the curve). Scaling the local flow hydrograph and the reservoir inflow hydrograph by the same factor can result in local flow becoming increasingly rare in relation the reservoir inflow frequency. For example, scaling a specific flood by a factor (that originally had 0.04 ACE reservoir inflow frequency and 0.10 ACE local flow frequency) might result in a reservoir inflow and coincident local flow that are both equivalent to a 0.01 ACE event. This can change the dynamics of simulated floods as opposed to what might really happen in nature. Depending upon the watershed, SPK feels its current method could result in conservative estimates of local flow runoff.

The two issues above may have a cancelling effect upon one another, the first being less conservative and the last one being too conservative. Further sensitivity analyses or refinement of the hydrology could be done in PED phase to assess the above concerns. For the feasibility study, it is currently recommended that the period of record assigned to the Mormon Slough at Bellota gage in the FDA program be 52 years (which is half the unregulated frequency curve period of record of 104 years at this location).   This 52 year period of record is also applicable to points downstream of the Bellota gage because 1) much of the downstream watershed has levees 2) there are only a few locations where additional local flow enters 3) the bulk of the water in the levees comes from upstream of Bellota.

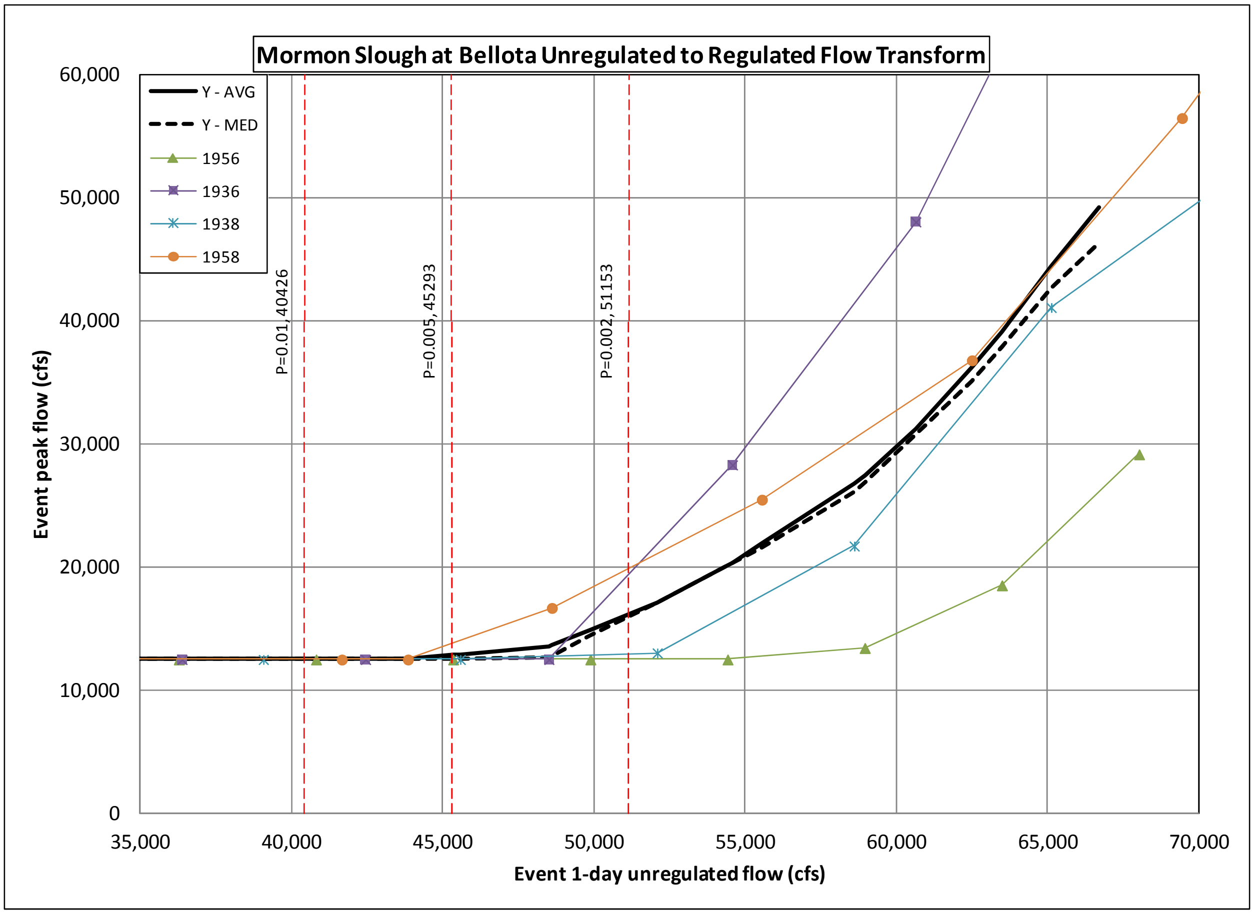


Figure 8. Unregulated 1-Day Flow to Regulated Peak Flow Transform at Bellota.

N-probability Events				
1/AEP	**AEP**	**Unregulated cfs**	**AVG transform**	**MEDIAN transform**
50	0.02	35,185	12,500	12,500
100	0.01	40,426	12,500	12,500
200	0.005	45,293	12,818	12,500
500	0.002	51,153	16,188	15,961

Table 7: 1-day Unregulated Flow and Regulated Peak Flow Comparison at Bellota.

Note: The median transform was adopted for Bellota as it appears to better fit the scaled event traces.

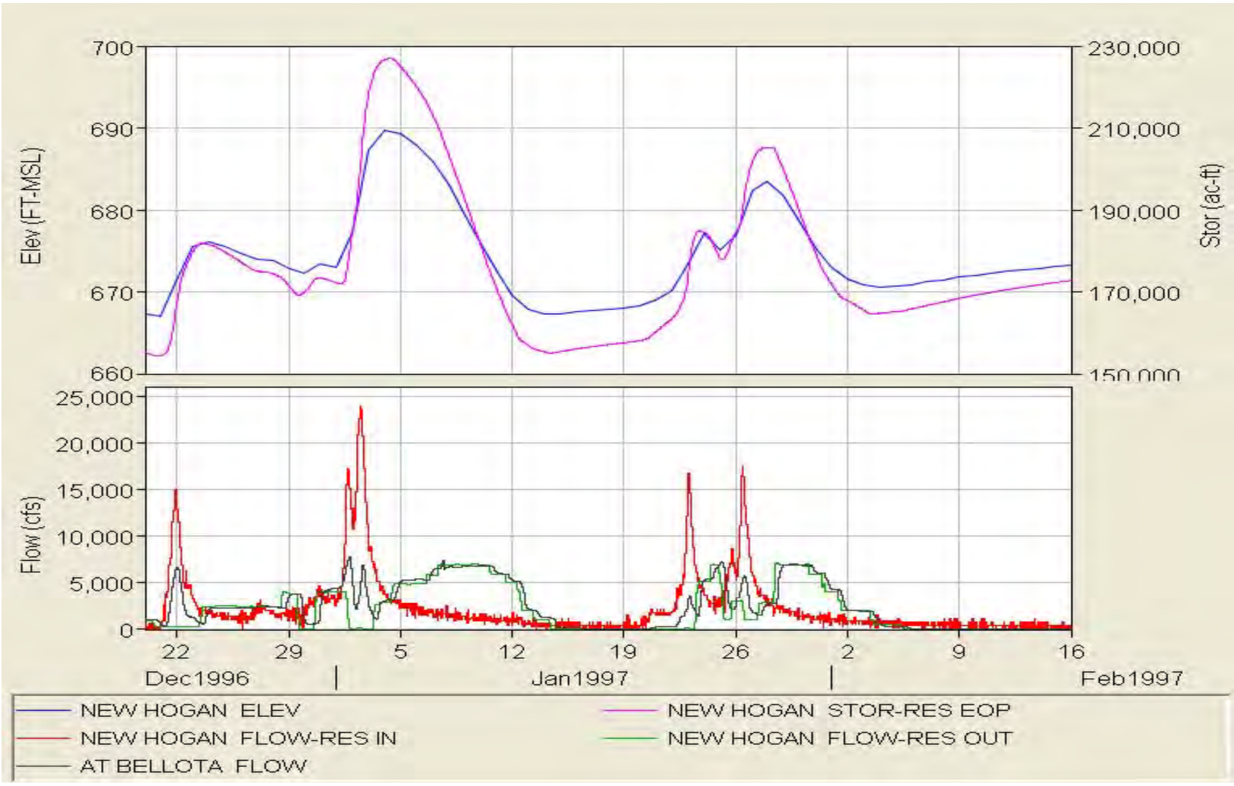


Figure 9. Actual operation of New Hogan dam during the 1997 flood event.

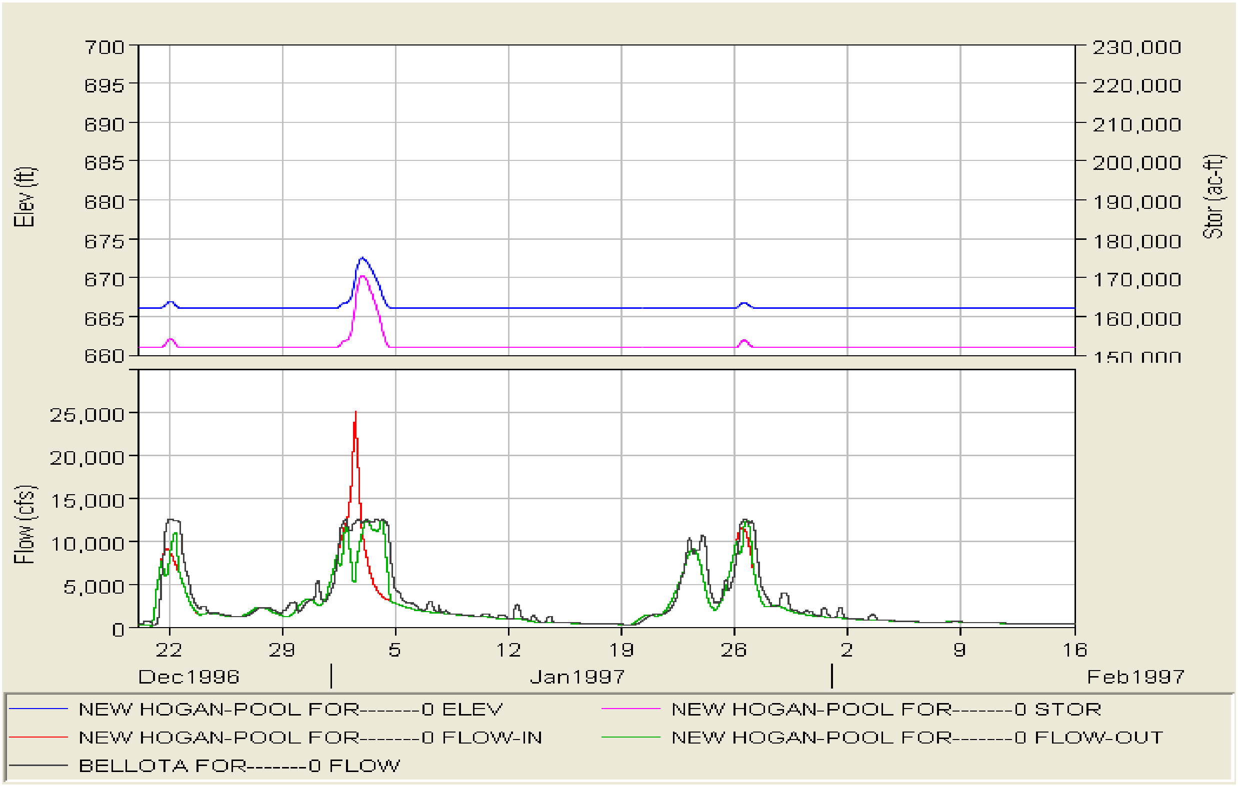


Figure 10. Simulated operation of New Hogan dam for the 1997 flood event.

Bellota n-Day Max Flows		Peak	1day	3day	7day	15day	1/Prob
No.	Prob	Y-Axis	Y-Axis	Y-Axis	Y-Axis	Y-Axis	
1	0.9583	738	105	91	64	62	1.04
2	0.9167	959	506	299	187	108	1.09
3	0.8750	1,284	617	387	216	173	1.14
4	0.8333	1,297	692	502	315	175	1.20
5	0.7917	1,404	1043	586	319	202	1.26
6	0.7500	1,463	1131	734	385	219	1.33
7	0.7083	2,144	1176	760	422	234	1.41
8	0.6667	2,186	1239	776	423	267	1.50
9	0.6250	2,228	1259	804	433	279	1.60
10	0.5833	2,343	1791	891	604	348	1.71
11	0.5417	3,016	1832	1,120	639	361	1.85
12	0.5000	3,515	2491	2,400	2,144	1,527	2.00
13	0.4583	4,439	3309	3,055	2,530	1,575	2.18
14	0.4167	4,501	3895	3,579	2,691	2,396	2.40
15	0.3750	5,111	3978	3,701	3,168	2,481	2.67
16	0.3333	6,820	4108	3,793	3,449	2,923	3.00
17	0.2917	7,833	6915	6,740	4,916	3,260	3.43
18	0.2500	9,499	7635	6,977	5,160	3,350	4.00
19	0.2083	9,514	7647	7,138	6,050	4,509	4.80
20	0.1667	9,519	7938	7,277	6,067	4,786	6.00
21	0.1250	9,635	8071	7,996	6,104	4,991	8.00
22	0.0833	9,876	8522	8,021	6,919	5,288	12.00
23	0.0417	10,602	9266	9,145	7,891	5,475	24.00

Interpolated Values							**1/Prob**
No.	**AEP**	**Peak**	**1day**	**3day**	**7day**	**15day**	
12	0.500	3515	2491	2400	2144	1527	2
19-20	0.200	9515	7702	7164	6053	4562	5
20-21	0.100	9529	8527	7560	6102	5345	10
22-23	0.040	10642	9307	9206	7943	5485	25
24	0.020	12,500	10,300	10,300	9,400	7,800	50
25	0.010	12,500	11,400	11,300	10,900	10,100	100
26	0.005	12,500	12,400	12,400	12,400	12,400	200
27	0.002	16,000	13,500	13,100	13,000	12,500	500

Values in Yellow are from Transform Curve and Table

Table 8. Peak, 1-, 3-, 7-, and 15-day Flows for Mormon Slough at Bellota from historic graphical curve.

Note: 0.50 to 0.04 ACE values derived from graphical curve of 1988 to 2010 water year data. 0.02 to 0.002 ACE highlighted in yellow are derived from reservoir simulations of scaled events

Regulated Peak Flow values and associated volumes: Mormon Slough at Bellota					
Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes1 (as average flow for given duration)			
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)
0.5	3,515	2,491	2,400	2,144	1,527
0.2	9,515	7,702	7,164	6,053	4,562
0.1	9,529	8,527	7,560	6,102	5,345
0.04	10,642	9,307	9,206	7,943	5,485
0.02	12,500	10,300	9,900	9,400	7,800
0.01	12,500	11,400	11,300	10,900	10,100
0.005	12,500	12,400	12,200	12,000	11,300
0.002	16,000	13,500	13,100	13,000	12,500

0.5 to 0.04 ACE: Peak & volume based on graphical curves from historic data
0.02 to 0.002 ACE: Peak based on Unreg. To Regulated Transform (Figure 8).
0.005 & 0.002 ACE event volumes from DFC's regulated peak to volume regression eqtns
0.02 & 0.01 ACE event volumes adjusted/smoothed to get consistency between
0.04 to 0.005 ACE events.

Table 9. Regulated Peak Flows and Associated Volumes for Mormon Slough at Bellota.

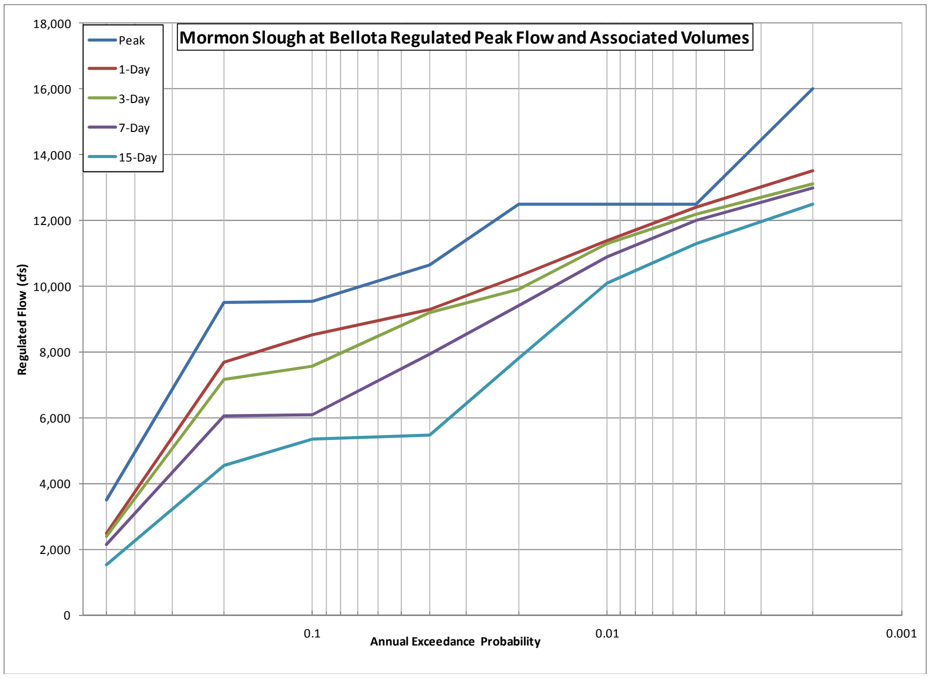


Figure 11. Regulated Peak Flow and Associated Volumes at Mormon Slough at Bellota.

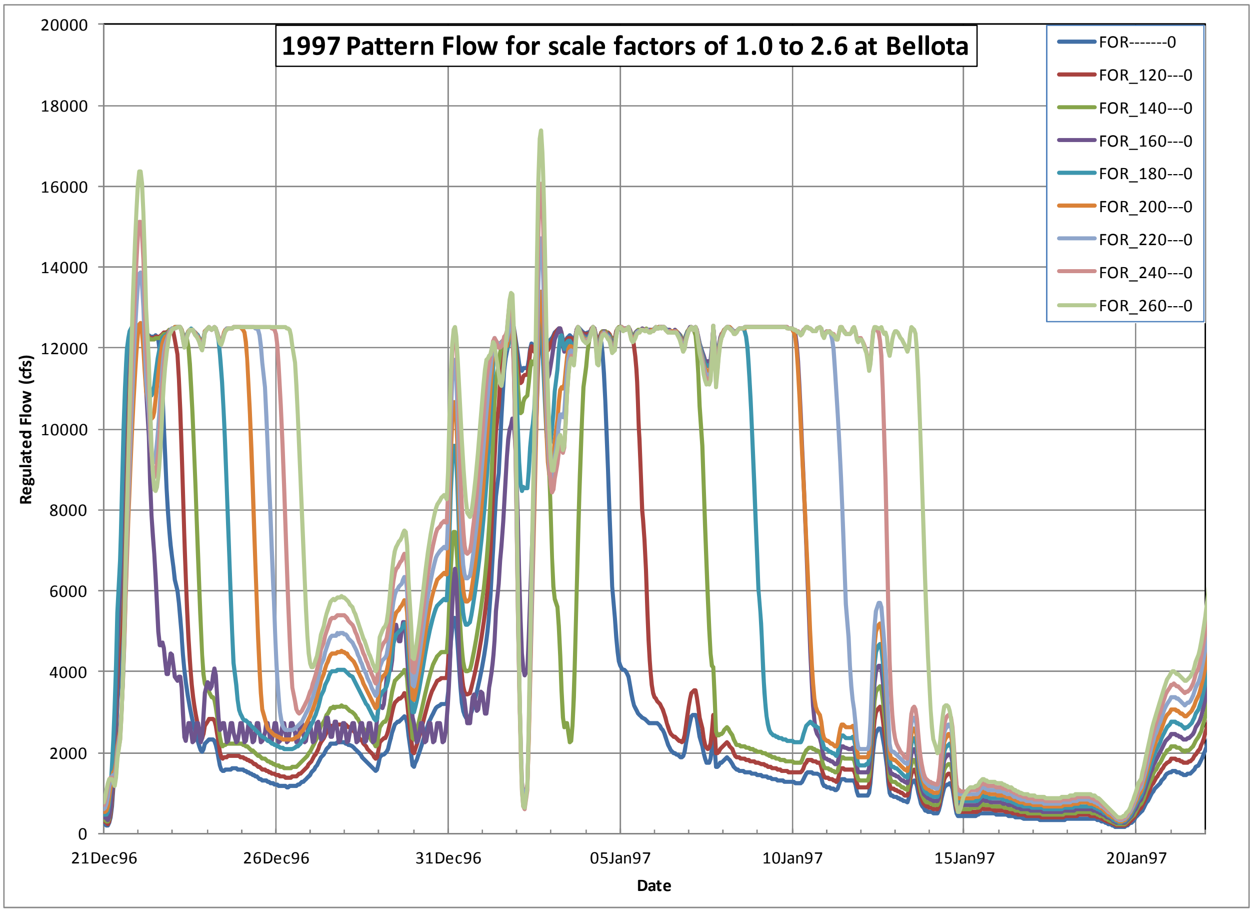


Figure 12. 1997 Pattern Flows for scale factors from 1.0 to 2.6 at Bellota

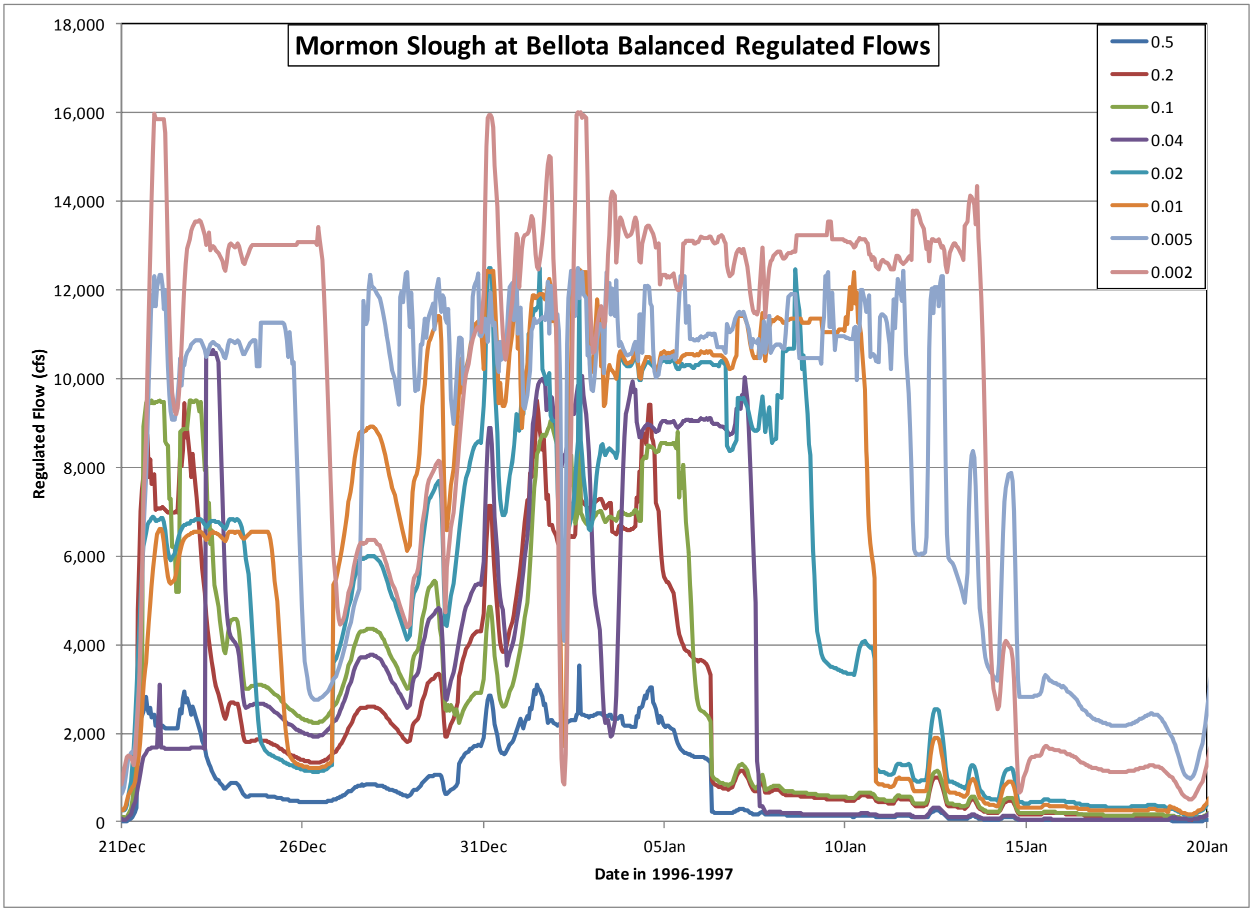


Figure 13. Final Balanced 1997 Pattern Hydrographs at Bellota

Associated files: In
In Corps directory:
W:\Studies\SJQ-020\LSJQR\Working Files\RegulatedFlows\NEW_Data

Filenames:
MSB-RegFlowFreq-1997Event-Hydrographs-30Jan2012.xlsx,

Bellota_TRANSFORM TEMPLATE – DRAFT 30Jan2012.xlsm,

Reconstruct-DFCE_Table4-30Jan2012.xlsx,

Ratios-BellotaLocal-to-ResIn&Cosgrove.xlsx,

Bellota-nday-GraphicalFit-01Feb2012.xlsx,

And this file: LSJR-FS-RegulatedFlows-07Feb2012.docx.

Appendix 1 - Attachment 1

Lower San Joaquin River Feasibility Study
Calaveras River above Bellota
Hydrologic Analysis

US Army Corps
of Engineers.

Sacramento District

07 April 2014

Lower San Joaquin River feasibility study: Calaveras River frequency analysis and hydrographs

June 20, 2011

US Army Corps of Engineers Sacramento District
W91238-09-D-0004, TO 0004

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Engineer's certification

I, Michael Konieczki, hereby certify on 6/20/2011 that I am a professional engineer licensed in the state of California and that the accompanying report was prepared by me or under my supervision.

REGISTERED PROFESSIONAL ENGINEER
MICHAEL V. KONIECZKI
No. 74357
Exp. 6/30/2013
CIVIL
STATE OF CALIFORNIA

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Tables

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Executive summary

Situation

In the lower San Joaquin River feasibility study (LSJR FS) the Sacramento District of the US Army Corps of Engineers (Corps) and the San Joaquin Area Flood Control Agency (SJAFCA) are studying alternative flood risk reduction measures that will provide protection against a flood with a probability of exceedence in any given year equal 0.005 (i.e., a "200-year flood").

The LSJR FS includes hydrologic analyses of the study region. This same region is also being studied in conjunction with a separate project to map the floodplains adjacent to the federal-state levee system in the Central Valley. Because the products of the various hydrologic analyses being conducted in the lower San Joaquin River basin will be used for several purposes by multiple agencies and stakeholders, the firms and agencies involved are using consistent analytical procedures and methods where possible. These procedures are specified in the *Sacramento and San Joaquin river basins: Procedures for hydrologic analysis* (hereinafter, *Procedures document*) and the *Central Valley hydrology study (CVHS): Technical procedures document* (hereinafter, *Technical procedures document*). Attachment 1 provides a table that explains how the procedures detailed in the present document align with the procedural steps detailed in the *Procedures document* and the *Technical procedures document*.

In this report we detail our hydrologic analyses at 2 sites on the Calaveras River: (1) New Hogan Reservoir, and (2) New Hogan's operation point at Bellota. These sites are shown in Figure 1.

Tasks

Our tasks were to: (1) develop a regulated flow-frequency curve and associated volumes at each location, and (2) derive an "expected" outflow hydrograph at New Hogan Reservoir.

Actions

To complete the tasks above, we:

- Developed unregulated volume-frequency curves at New Hogan Reservoir and Bellota following the procedures in *Guidelines for determining flood flow frequency, Bulletin 17B* (IACWD 1982) and EM 1110-2-1415 (USACE 1993) and using a regional skew provided by the Corps.
- Simulated reservoir releases and routed historical and scaled floods, including local flows, on the Calaveras River using an HEC-ResSim model provided by the Corps.
- Fitted, at each location, flow transforms to the event maxima datasets identified from the unregulated flow and simulated release time series.
- Developed, at each location, a regulated flow-frequency curve and associated volumes by applying the flow transforms.
- Developed "expected" outflow hydrographs for New Hogan Reservoir for 8 flood frequencies: p=0.5, p=0.2, p=0.10, p=0.05, p=0.02, p=0.01, p=0.005 and p=0.002. (Here the term expected hydrograph refers to a

New Hogan Reservoir outflow hydrograph with a peak flow that matches the regulated flow-frequency curve and with associated volumes matching those from the family of characteristic curves corresponding to the given regulated peak flow.)

Results

The results of our analysis include:

- Unregulated volume-frequency curves for New Hogan Reservoir (as shown in Figure 2).
- Unregulated volume-frequency curves for the Calaveras River at Bellota (as shown in Figure 3).
- Unregulated-regulated flow transform for New Hogan Reservoir (as shown in Figure 4).
- Regulated flow-frequency curve and associated volumes for New Hogan Reservoir (as shown in Table 1 and in Table 2).
- Unregulated-regulated flow transform for the Calaveras River at Bellota (as shown in Figure 5).
- Regulated flow-frequency curve and associated volumes for the Calaveras River at Bellota (as shown in Table 3 and in Table 4).
- Expected hydrograph properties for New Hogan Reservoir. (Note: these are the same values shown in Table 1).

In addition, these intermediate values and information are included with the original report on DVD:

- HEC-DSS time series of the floods-of-records.
- HEC-DSS time series of the scaled historical floods.
- HEC-DSS time series of developed local flows below New Hogan Reservoir (detailed in Attachment 2).
- The tabulated event maxima datasets for the 2 analysis sites.
- Simulated reservoir releases and routed flows from the HEC-ResSim reservoir simulation model.
- Tabulated unregulated-regulated flow transforms for the 2 analysis sites.
- Tabulated families of regulated characteristic curves for the 2 analysis sites.

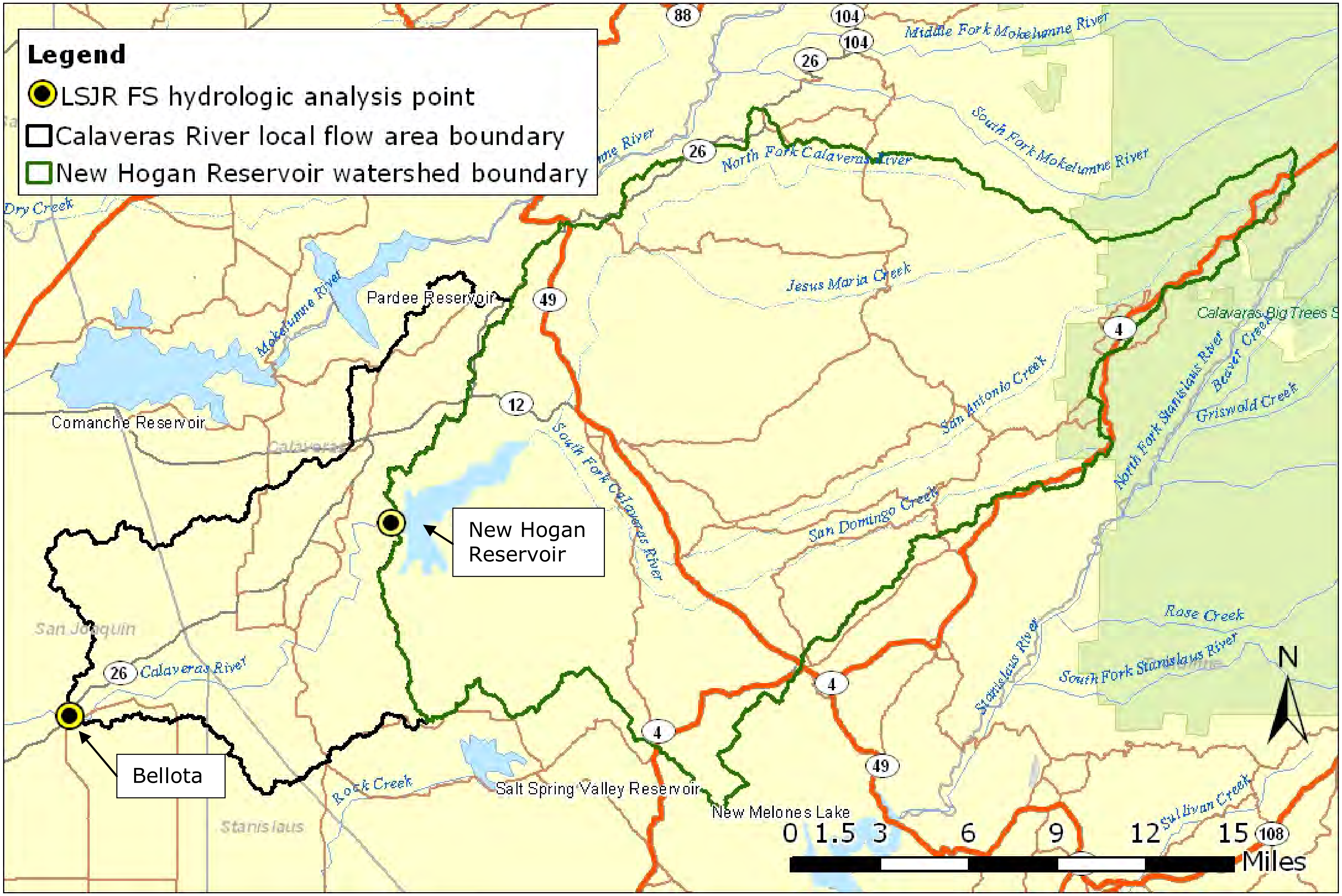
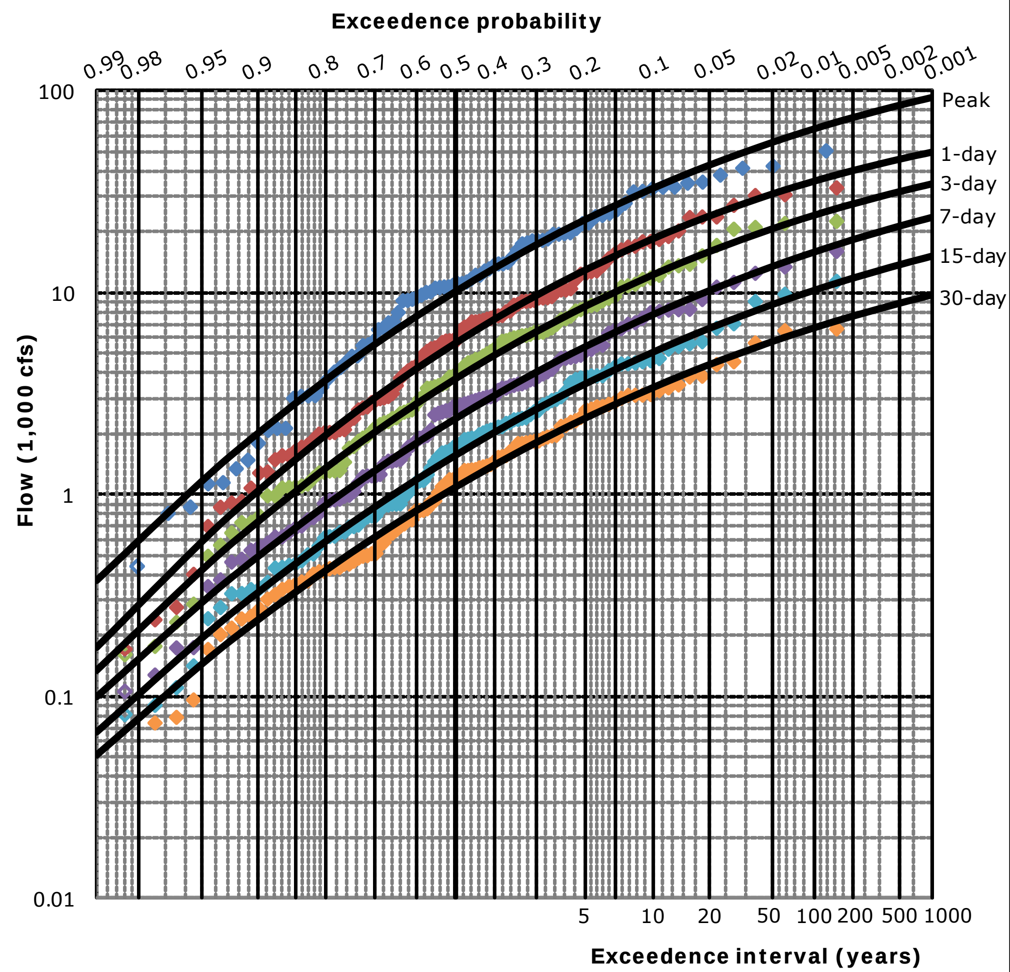


Figure 1. Calaveras River study area



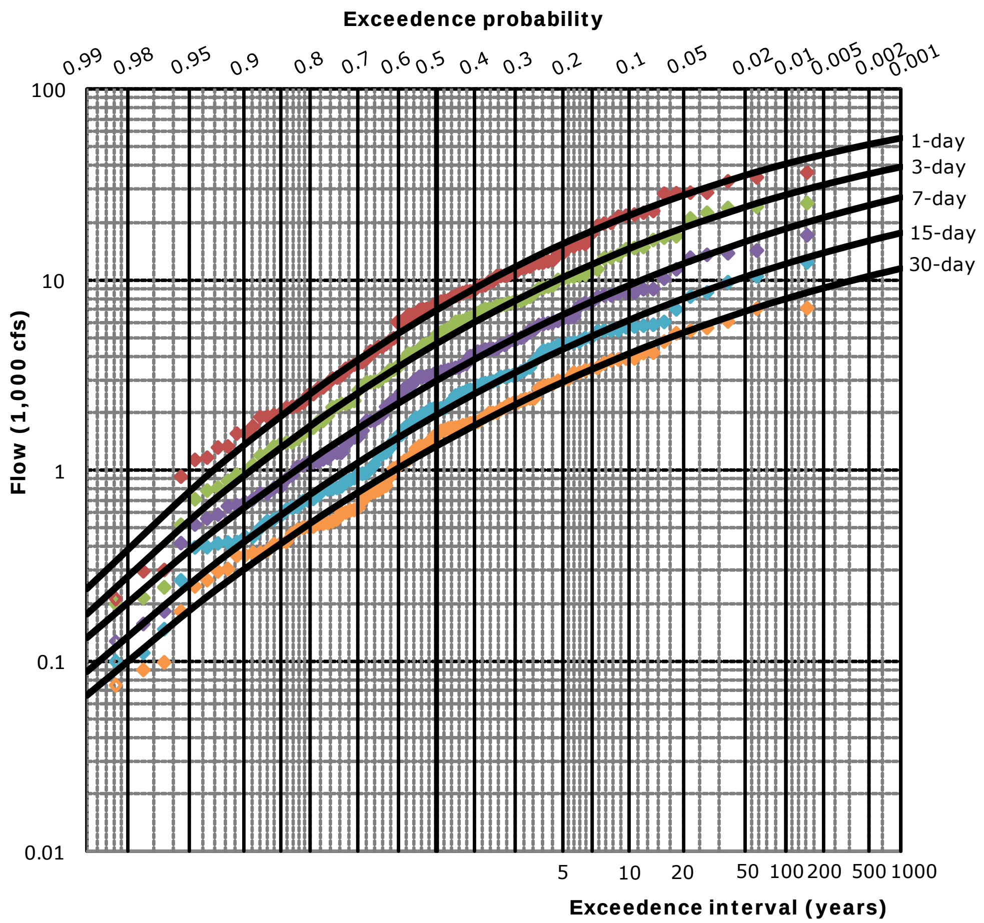
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.946	0.482	-0.727
1-day	3.685	0.501	-0.794
3-day	3.518	0.487	-0.732
7-day	3.324	0.477	-0.651
15-day	3.146	0.473	-0.656
30-day	2.988	0.457	-0.659

Notes:

- Median plotting positions.
- Drainage area: 363 sq. miles.
- Period of systematic record: 1907-2010.
  (Peak flow data intermittent 1930-2010).
- Record lengths
  Peak flows: 86 years.
  Volumes: 104 years.
- Regional skew values developed by USGS.
- Hollow points are censored events.

Figure 2. Unregulated frequency curves: New Hogan Reservoir



Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
1-day	3.775	0.482	-0.810
3-day	3.608	0.475	-0.753
7-day	3.417	0.464	-0.666
15-day	3.240	0.461	-0.671
30-day	3.079	0.448	-0.668

Notes:

- Median plotting positions.
- Drainage area: 473 sq. miles.
- Period of systematic record: 1907-2010.
- Record length: 104 years.
- Regional skew values developed by USGS.
- Hollow points are censored events.

Figure 3. Unregulated frequency curves: Calaveras River at Bellota

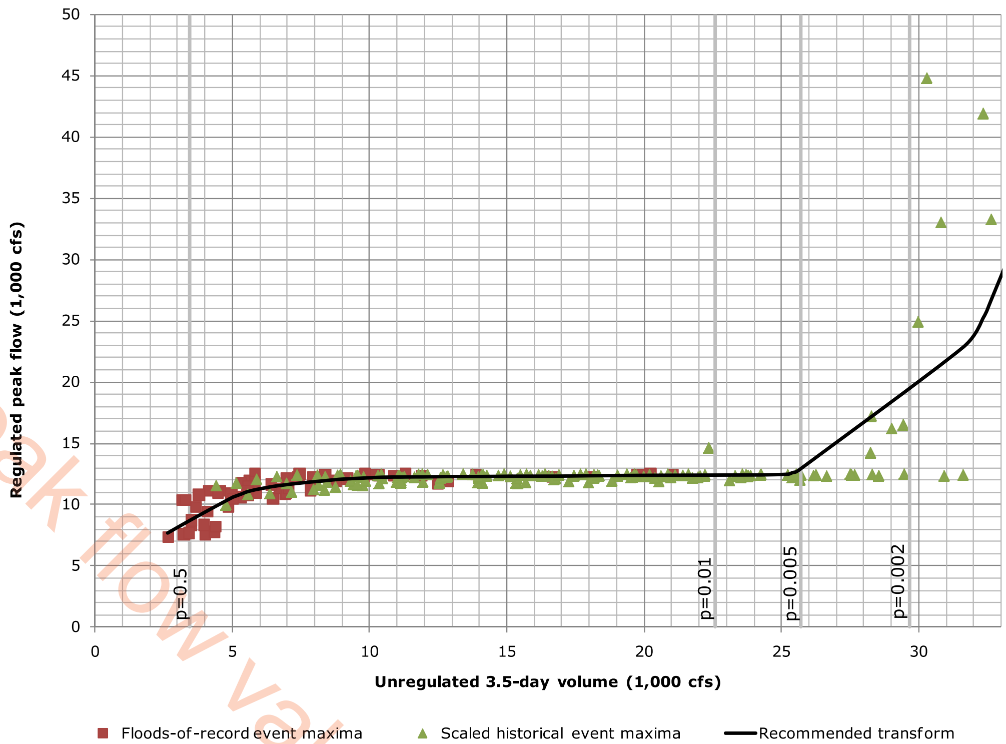


Figure 4. Unregulated-regulated flow transform: New Hogan Reservoir

Table 1. Regulated peak flow-frequency quantiles: New Hogan Reservoir

Annual exceedence probability (1)	1/annual exceedence probability (2)	Regulated peak flow (cfs) (3)
0.500	2	8,664
0.200	5	11,812
0.100	10	12,214
0.050	20	12,266
0.020	50	12,334
0.010	100	12,367
0.005	200	12,903
0.002	500	19,555

Table 2. Regulated peak flow values and associated volumes: New Hogan Reservoir

Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes[1] (as average flow for given duration)				
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)	30-day (cfs) (7)
0.500	8,664	6,212	4,188	2,720	1,843	1,199
0.200	11,812	11,625	10,634	7,457	4,994	3,096
0.100	12,214	12,107	11,582	9,098	5,909	3,963
0.050	12,266	12,140	11,607	9,312	6,032	4,157
0.020	12,334	12,283	11,880	10,275	7,045	5,120
0.010	12,367	12,300	11,916	10,459	7,411	5,263
0.005	12,903	12,900	12,893	12,876	12,026	9,283
0.002	19,555	19,555	19,549	17,462	12,445	9,463

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at New Hogan Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

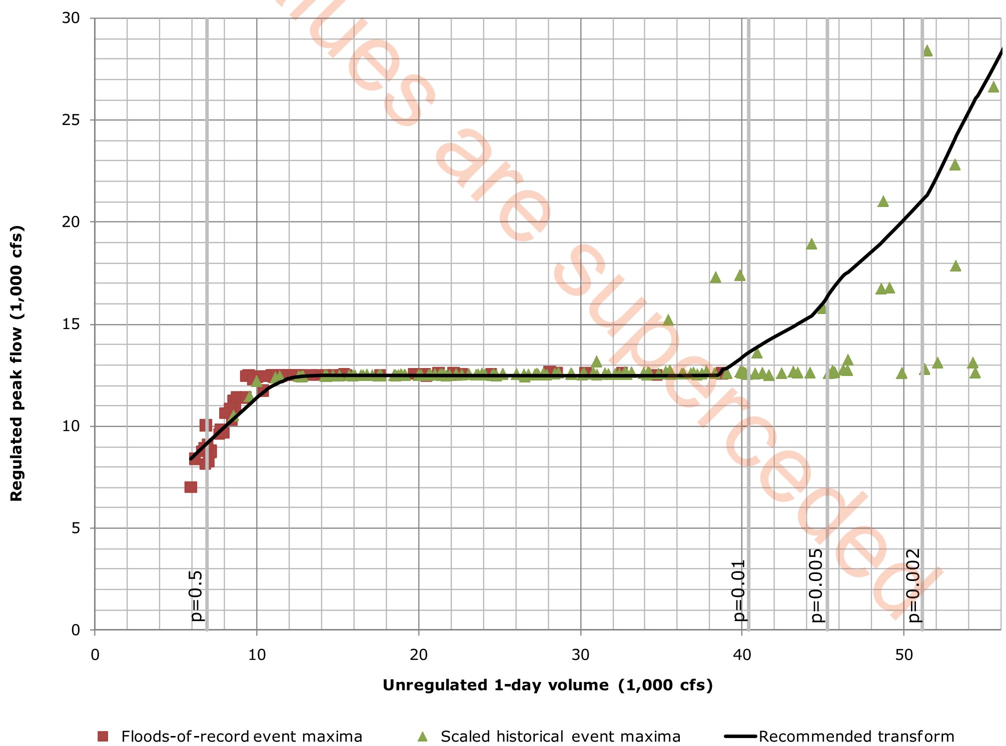


Figure 5. Unregulated-regulated flow transform: Calaveras River at Bellota

Table 3. Regulated peak flow-frequency quantiles: Calaveras River at Bellota

Annual exceedence probability (1)	1/annual exceedence probability (2)	Regulated peak flow (cfs) (3)
0.500	2	9,163
0.200	5	12,500
0.100	10	12,500
0.050	20	12,500
0.020	50	12,500
0.010	100	13,634
0.005	200	16,409
0.002	500	21,107

Table 4. Regulated peak flow values and associated volumes: Calaveras River at Bellota

Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes[1] (as average flow for given duration)				
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)	30-day (cfs) (7)
0.500	9,163	7,271	4,852	3,163	2,127	1,372
0.200	12,500	12,500	12,500	12,500	12,500	10,000
0.100	12,500	12,500	12,500	12,500	12,500	10,000
0.050	12,500	12,500	12,500	12,500	12,500	10,000
0.020	12,500	12,500	12,500	12,500	12,500	10,000
0.010	13,634	13,174	13,141	12,545	12,515	10,001
0.005	16,409	13,367	13,300	13,300	12,553	10,002
0.002	21,107	15,106	15,106	13,930	12,631	10,005

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at New Hogan Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Watershed description

The watershed that is the subject of this report—the Calaveras River basin—is part of the lower San Joaquin River basin. It is located in Calaveras, San Joaquin, and Stanislaus counties. Located on Calaveras River approximately 28 miles upstream of Stockton, CA, is New Hogan Reservoir, a multipurpose facility with water supply, recreation, and flood control requirements.

The 707 mi^2 Calaveras River basin is shown in Figure 6. The north and south forks of the Calaveras River meet just east of New Hogan Reservoir and continue flowing into the reservoir. The basin comprises 3 major areas:

- The area above New Hogan Reservoir, which includes 363 mi^2 of relatively low-lying area on the western slopes of the Sierra Nevada. Elevations range from 550 ft at the dam to approximately 6,000 ft at the highest point.
- The 110 mi^2 area between New Hogan Reservoir and the downstream operation point at Bellota (the bifurcation of the Old Calaveras River and Mormon Slough approximately 18 miles downstream of the reservoir). The elevation at Bellota is approximately 130 feet.
- The remaining 234 mi^2 area of the Calaveras River and Mormon Slough watershed from Bellota to the confluence with the San Joaquin River. This portion of the watershed is low and flat with little topographic relief. Note: hydrological analysis of this region is being completed by other consultants and agencies and is therefore beyond the scope of the analysis described here.

The channel capacity downstream of New Hogan Reservoir is 12,500 cfs and the reservoir operates to limit flow to this value downstream of the dam and at Bellota (USACE 1983). A control structure exists at Bellota to divert the majority of flows into Mormon Slough. Downstream of this structure lies the Old Calaveras River channel, which is overgrown with vegetation. Flow is diverted into the Old Calaveras River when flow at Bellota reaches 13,500 cfs (USACE 1983).

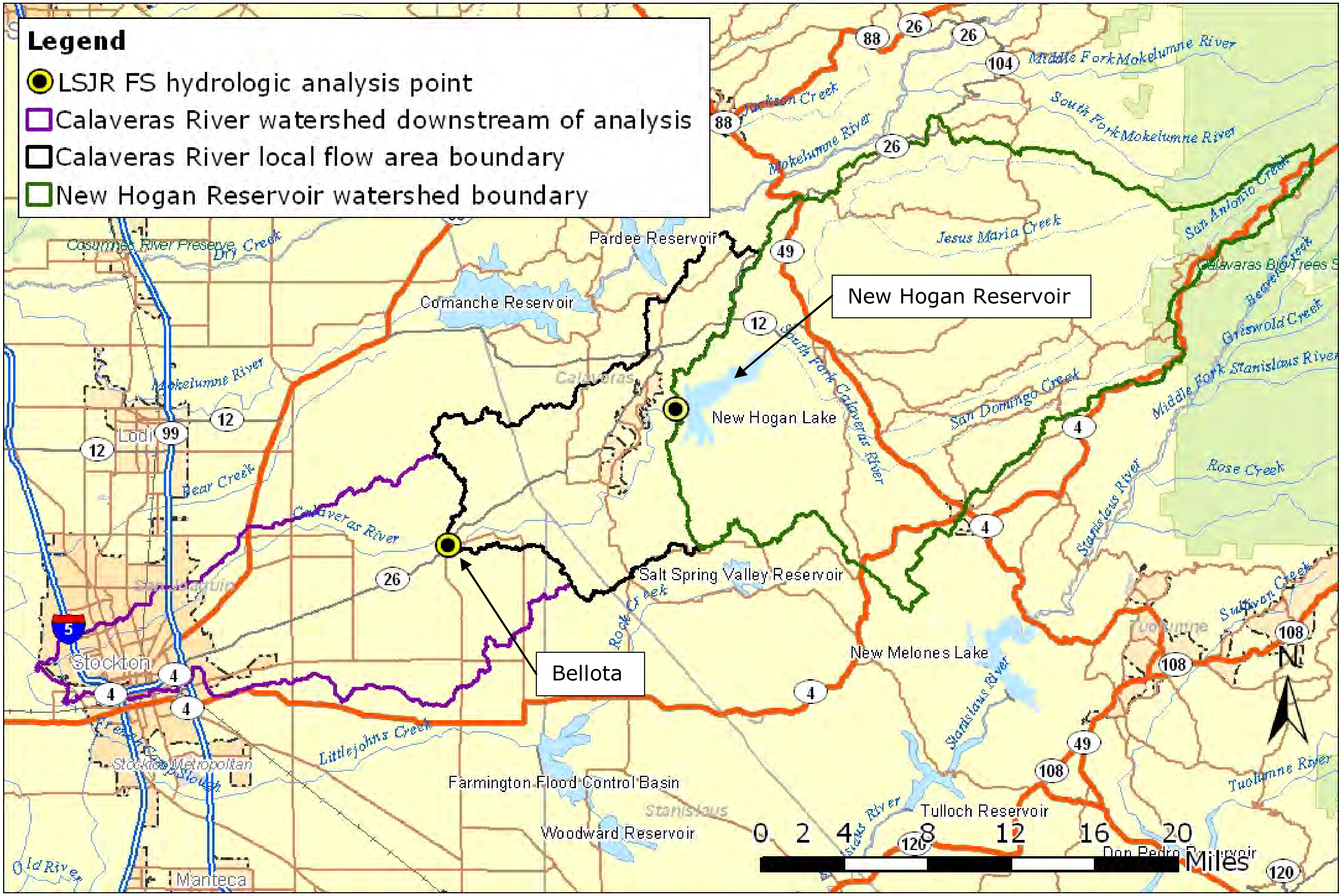


Figure 6. Lower San Joaquin River feasibility study area: Calaveras River

Analysis procedure

Overview of CVHS procedure

The primary tasks for the CVHS are described in the *Procedures document*. More detail for these tasks is provided in the *Technical procedures document*. As a review of those tasks and to provide context for the procedures used in this analysis, here we summarize the procedure steps and categorize them into 2 groups. They are:

- Group 1. Unregulated frequency analysis at selected points. This comprises *Procedures document* Task 1, Task 2 (reservoir simulation models), Task 3, and Task 4. (References throughout this report to numbered tasks use numbers from the *Procedures document*.)
- Group 2. Description of the effects of the regulation (flood control) system to allow conversion of the unregulated frequency curves to regulated flow-frequency curves at the same selected points. This comprises *Procedures document* Task 2 (channel routing models), Task 5, Task 6, and Task 7.

Group 1 focuses on completing a frequency analysis to characterize the annual exceedence probability of a given flow (unregulated). Thus, all statements of probability originate here.

Group 2 reflects the impact of regulation in the system. This second group accounts for various historical storm distributions and reservoir operations, with an emphasis on large events.

Application to the lower San Joaquin River feasibility study

In Figure 7, we illustrate the general work flow of the analysis procedure as applied to the LSJR FS. In this document we note before each analysis step the corresponding CVHS procedures task applicable, if any.

For unregulated frequency analysis for the 2 sites on the Calaveras River, New Hogan Reservoir and Bellota, we:

- (Task 1) Obtained reservoir inflow and streamgage data for use in developing the unregulated flow time series from the Corps.
- (Task 2) Obtained accepted reservoir simulation and channel routing models from the Corps.
- (Task 3) Developed unregulated flow time series at each location corresponding to a period-of-record of floods. This step includes the development of local flows for the ungaged area between New Hogan Dam and Bellota.
- (Task 4) Computed and adopted unregulated 1-, 3-, 7-, 15-, and 30-day volume-frequency curves at each location. Note: we developed peak unregulated flow-frequency curves for New Hogan Reservoir for completeness; they are not required for this analysis.

For regulated system analysis for the 2 sites on the Calaveras River we:

- (Task 5) Developed regulated flow time series at each location by simulating and routing reservoir releases. Here, historical and scaled historical events were used in development of the time series.

- (Task 6) Fitted flow transforms. First, the unregulated and corresponding regulated event maxima datasets were identified (these are data points to which the transforms were fitted). Then, the critical duration of each analysis location was determined using these series. The flow transforms were then developed by fitting curves to the event maxima datasets. Note here, the term flow transforms refers to: (1) the unregulated-regulated flow transform, and (2) the family of regulated characteristic curves.
- (Task 6.4) Applied flow transforms to develop a regulated peak flow-frequency curve and associate volumes for the 1-, 3-, 7-, 15-, and 30-day durations at each location.

For development of the expected hydrograph properties for New Hogan Reservoir outflows we identified the peak regulated flows and associated regulated volume-duration characteristics for 8 exceedence probabilities: p=0.5, p=0.2, p=0.1, p=0.05, p=0.02, p=0.01, p=0.005, and p=0.002.

Attachment 1 provides a table explaining how the procedures detailed here align with the procedural steps detailed in the *Procedures document* and the *Technical procedures document*.

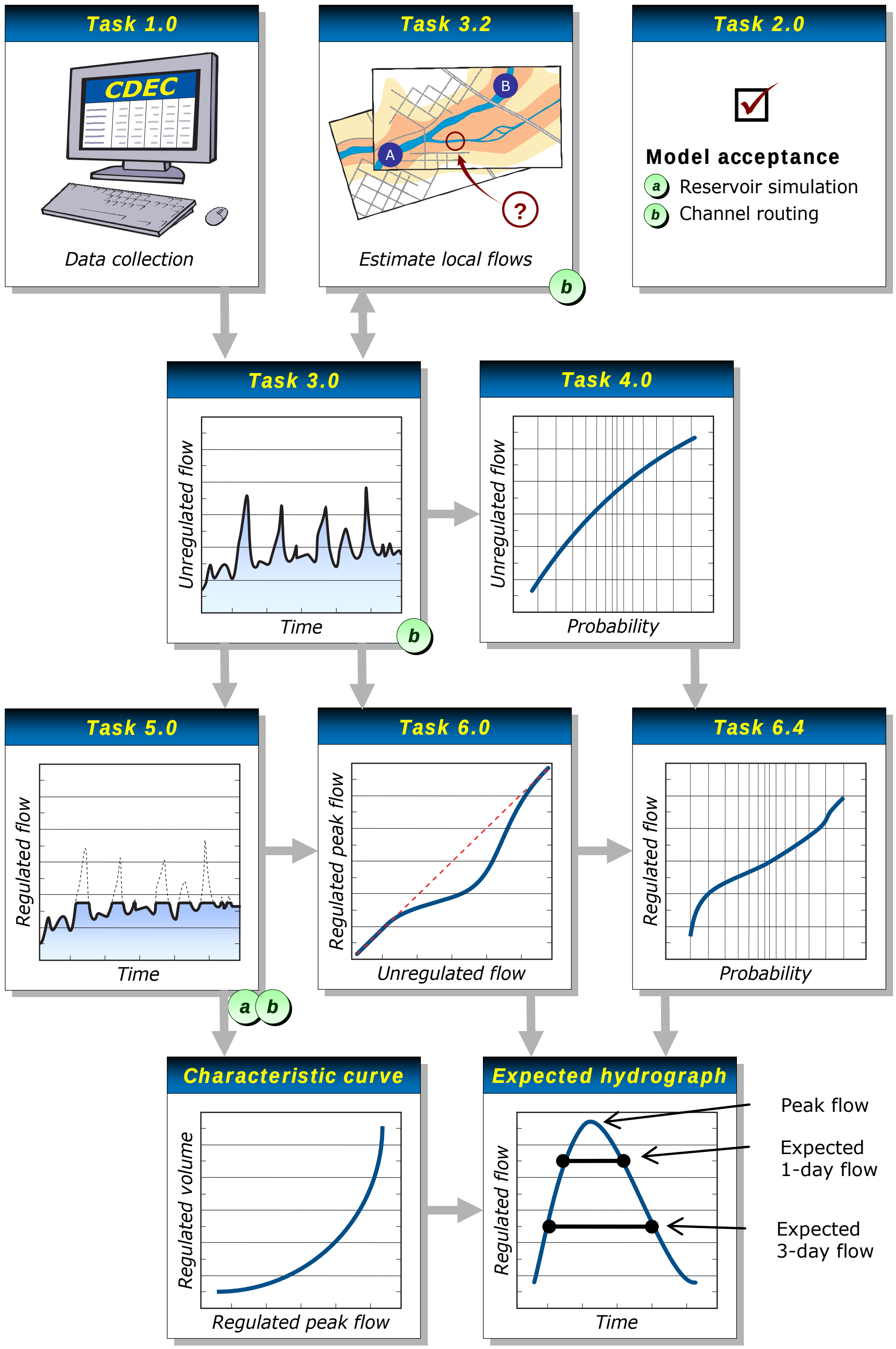


Figure 7. LSJR analysis procedure workflow

Unregulated flow time series development

We constructed unregulated flow time series at each analysis location in the study area and fitted unregulated volume-frequency curves to these series using procedures that are consistent with Corps guidance.

The locations most upstream at which we developed unregulated flow time series were the project reservoirs. Thus, for unregulated conditions, the reservoir inflows were needed.

For development of the unregulated flow time series downstream of the reservoir, a routing model was required to simulate the translation, attenuation, and combination of the unregulated flow hydrographs through the system. These flow hydrographs included the upstream boundary conditions (derived reservoir inflows) and intermediate area boundary conditions (estimated local flows). The routing yielded unregulated flow time series that served as the basis of: (1) the unregulated frequency analysis and (2) the unregulated-regulated flow transform.

For this analysis, we developed an unregulated flow time series for the 2 analysis locations on the Calaveras River by:

- (Task 1) Obtaining daily unregulated reservoir inflow time series developed by the Corps.
- (Task 3.2) Developing local flow time series for the area between New Hogan Reservoir and the reservoir's control point at Bellota (shown in Figure 8).
- (Task 3.3) Completing the unregulated flow time series at each analysis point.

Obtain daily reservoir inflow

We obtained the daily unregulated reservoir inflows from the Corps. The Corps developed the daily unregulated reservoir inflow time series for New Hogan Reservoir using the continuity equation, in which, for a given time step, the average inflow equals the outflow plus the change in reservoir storage. For the calculation of these inflows, the source of the observed reservoir outflows and observed changes in storage was the Corps's database. By convention in the Central Valley, these calculations were completed on a 1-day time step, thus midnight to midnight values were used. This is consistent with the work completed for the *Sacramento and San Joaquin river basins comprehensive study* (Comp Study) completed in 2002 (USACE 2002).

Estimate local flow

For the Calaveras River, local flows needed to be estimated for the area between New Hogan Reservoir and Bellota, shown in Figure 8. The estimation approaches we used were:

- Option 1. Direct calculation of local flow using known releases from New Hogan Reservoir and the observed flows at Bellota, routing hourly flows as necessary. In the case of missing streamgage data, local flows values were interpolated as needed.
- Option 2. Estimation of local flows as:

$$Q_{Local} = 3.2\left(Q_{Cosgrove}\right) \quad (1)$$

where Q_{Local} is the local flow estimate for a given time, and $Q_{Cosgrove}$ is the observed flow at the Cosgrove Creek near Valley Springs, CA, streamgage. The Corps estimates local flows for the purpose of real-time reservoir operations using this option (John High, personal communication, 11/9/2009).

- Option 3. Estimation local flows as:

$$Q_{Local} = 0.226\left(Q_{NHG}\right) \quad (2)$$

where Q_{Local} is the local flow estimate for a given time, and Q_{NHG} is the unregulated inflow to New Hogan Reservoir. We developed this equation as detailed in Attachment 2.

In Table 5 we summarize the selected approaches for local flow estimation on the Calaveras River by water year. This flow represents the total local flow contribution at Bellota. We detail the development of the local flow time series on the Calaveras River in Attachment 2.

Table 5. Selected local flow estimation approaches for the area on the Calaveras River between New Hogan Reservoir and Bellota

Time period (water year) (1)	Time step (2)	Selected approach[1] (3)
1907-1929	Daily	Option 3: 0.226 times reservoir inflow.
1930-1969	Daily	Option 2: 3.2 times Cosgrove Creek flow.
1970-1987	Daily	Option 3: 0.226 times reservoir inflow.
1988	Daily	Option 1: directly calculate local flow.
1989	Daily	Option 3: 0.226 times reservoir inflow.
1990-1993	Daily	Option 1: directly calculate local flow.
1994-1995	Daily	Option 3: 0.226 times reservoir inflow.
1996-2009	Hourly	Option 1: directly calculate local flow.
2010	Daily	Option 2: 3.2 times Cosgrove Creek flow.

1. The approach listed is the predominant method for estimating local flows over the time period given. See Attachment 2 for further detail.

Complete unregulated flow time series

For the unregulated frequency analysis, we used the daily unregulated reservoir inflow time series provided by the Corps directly as the unregulated time series corresponding to New Hogan Reservoir. For the reservoir's operation point on the Calaveras River at Bellota, we combined the daily unregulated inflow time series with the estimated local flows by adding the 2 time series together. We did not route the unregulated reservoir inflows because: (1) synthesizing a shorter time step is not required for frequency analysis, and (2) the travel time between the reservoir and the operation point is approximately 7 hours, which is less than the 1-day time step of the inflows. In addition, there is little attenuation of flood peaks in this reach because of its length and channel geometry. We confirmed this by comparing observed releases from New Hogan Reservoir, observed flows on Cosgrove Creek, and observed flows on the Calaveras River at Bellota.

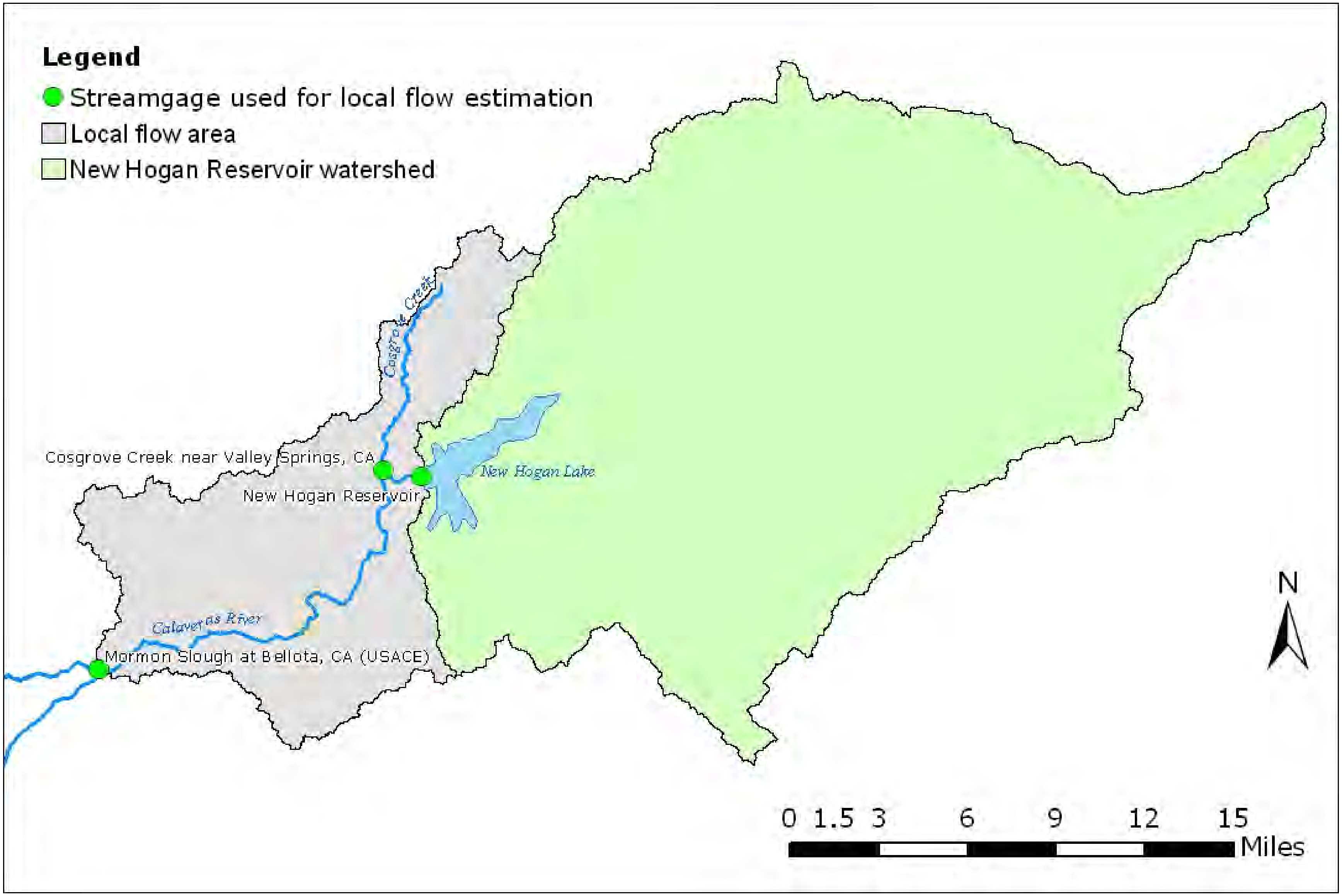


Figure 8. Calaveras River local flow area between New Hogan Reservoir and Bellota and study streamgages

Unregulated frequency analysis

Commonly accepted procedures to develop unregulated flow-frequency curves are specified in *Bulletin 17B* (IACWD 1982). The current standard-of-practice is to fit a Pearson III (LPIII) distribution to the logarithmic transforms of annual maximum series identified from streamgage data. Additional guidance for fitting frequency curves to volumes for a given duration is provided by EM 1110-2-1415 (USACE 1993).

For this analysis, we used the unregulated inflows to New Hogan Reservoir to develop such an annual maximum series. However, because we only had records of regulated flows on the Calaveras River at Bellota, we could not fit a frequency curve directly using this method. Thus, we used the synthesized unregulated flow time series at this location and fitted a volume-frequency curve to that series using procedures that are consistent with Corps guidance.

For this analysis we developed unregulated frequency curves following the procedures specified in *Bulletin 17B* (IACWD 1982), EM 1110-2-1415 (USACE 1993), and the current standards of practice. For each analysis location, we:

- Identified the annual maximum series.
- (Task 4.1) Calculated regional skew values for each duration of interest using relationships developed by the USGS.
- (Task 4.2) Fitted LPIII distributions to the annual maximum series following *Bulletin 17B* procedures and Corps guidance using the expected moment algorithm (EMA) enabled flow-frequency software PeakfqSA, version 0.937. This was developed by Tim Cohn of the USGS and is based on the USGS's flow-frequency software PeakFQ (Cohn 2007).
- Reviewed and adopted the curves, checking them for consistency and comparing them to previously accepted values.

Identify annual maximum series

We identified the annual maximum series by extracting, from the unregulated flow time series, the volumes associated with the 1-, 3-, 7-, 15-, and 30-day durations. This information is detailed in Attachment 3.

Note we developed a peak unregulated flow-frequency curve for New Hogan Reservoir for completeness; however this is not required for this analysis. The peak annual maximum series was provided by the Corps and is included in Attachment 3. In addition, we did not develop a peak flow-frequency curve for the Calaveras River at Bellota because the temporal resolution of the unregulated flow time series, 1 hour to as long as 1 day, is not an appropriate representation of instantaneous unregulated peak flow values.

Calculate regional skew values

For this analysis, we calculated regional skew values for the peak flows and 1-, 3-, 7-, 15-, and 30-day volumes using the relationships developed by the USGS (USGS 2010). In these relationships, the regional skew value is a function of the average basin elevation. The values calculated for each analysis location and duration of interest are shown in Attachment 4.

Fit frequency curves

To fit frequency curves to the annual maximum series we used: (1) the statistics of the logarithmic transforms of unregulated flow time series (mean, standard deviation, and skew), and (2) the regional skew values for the peak flow, and 1-, 3-, 7-, 15-, and 30-day calculated using relationships developed by the USGS (2010). The "at station" statistics were calculated using the EMA option in PeakfqSA.

We fitted the curves using a straightforward *Bulletin 17B* procedure in which all data points were included in the analysis. Low outliers were identified by the *Bulletin 17B* outlier test (implemented automatically by the program). The station statistics were then appropriately adjusted. This includes weighting the station skew and regional skew values by the inverse of their associated errors. This weighting procedure is included in *Bulletin 17B,* and the weighted skew is automatically calculated by the PeakfqSA software used here.

Review and adopt curves

After fitting, we reviewed the frequency curves for consistency and appropriateness. Specifically, we:

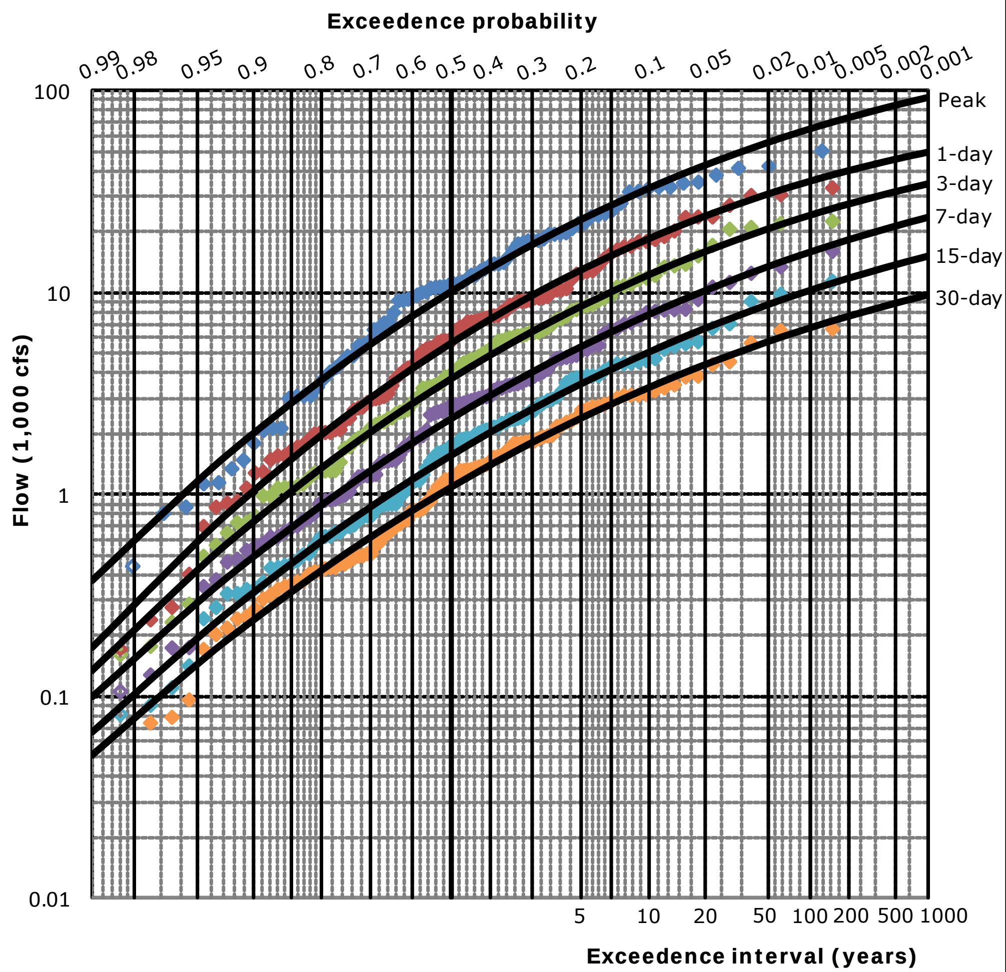
- Compared the curve of a given duration to the curves associated with the other durations at the same analysis location.
- Compared the curves at a given location to the curves at the other analysis location to check for consistency.
- Compared the curves to those published in the Comp Study.

We found the frequency curves on the Calaveras River were consistent between durations at each location. The curves do not "cross," and flow quantiles for a given duration at the downstream location are greater than those of the upstream location, as would be expected.

As a comparison, we considered the volume-frequency curves developed for New Hogan Reservoir in the Comp Study (USACE 2002). The annual maximum series in the Comp Study ended in 1997.

We also found that the flow quantiles of the curves fitted here and those of the Comp Study differ between the 2 sets of volume-duration curves by only 1%-13%. The greatest differences (of 8%-13%) are in the 1-day volume quantiles. The 3-day and 7-day volume quantiles differ by only 1% to 5%. Peak flow-frequency curves varied by as much as 9% because of the increased number of large events included in this analysis as compared to the Comp Study.

We adopted the unregulated frequency curves for the 2 analysis locations, New Hogan Reservoir and Bellota, shown in Figure 9 and Figure 10. These are the curves that use the automatic implementation of the *Bulletin 17B* outlier test. The detailed parameters used to fit these curves are included in Attachment 4.



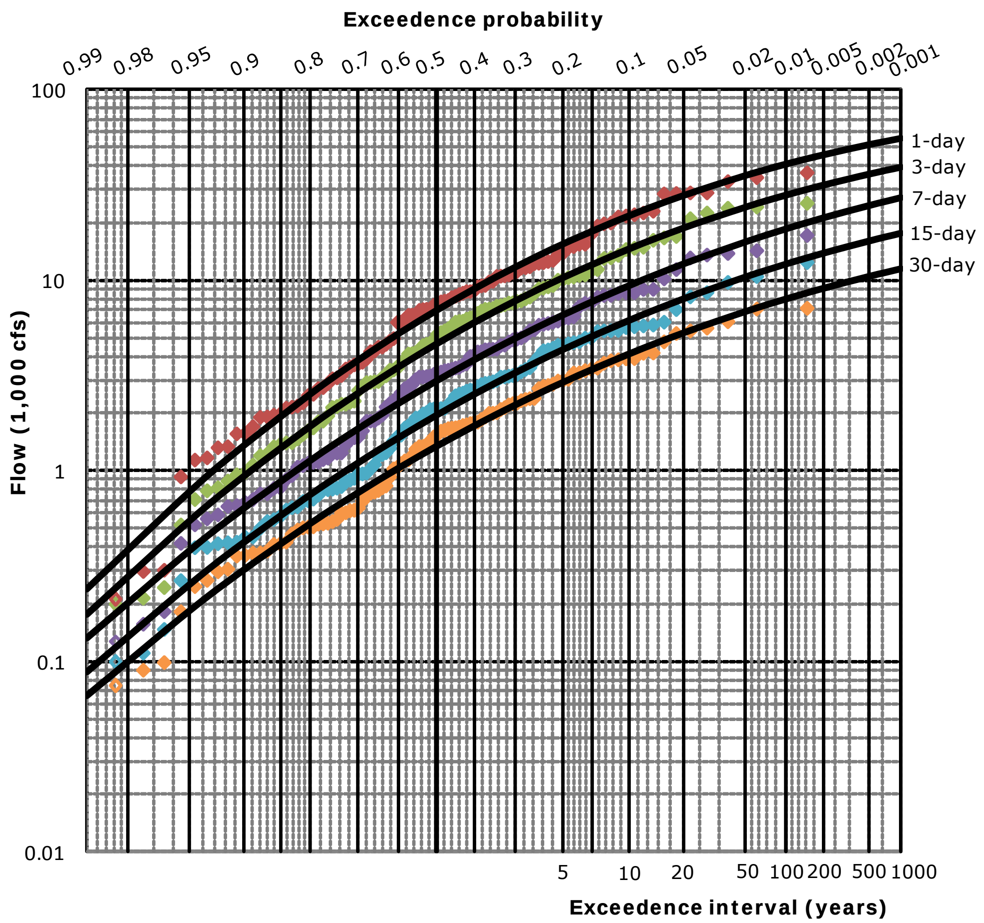
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.946	0.482	-0.727
1-day	3.685	0.501	-0.794
3-day	3.518	0.487	-0.732
7-day	3.324	0.477	-0.651
15-day	3.146	0.473	-0.656
30-day	2.988	0.457	-0.659

Notes:

- Median plotting positions.
- Drainage area: 363 sq. miles.
- Period of systematic record: 1907-2010.
  (Peak flow data intermittent 1930-2010).
- Record lengths
  Peak flows: 86 years.
  Volumes: 104 years.
- Regional skew values developed by USGS.
- Hollow points are censored events.

Figure 9. Unregulated frequency curves: New Hogan Reservoir



Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
1-day	3.775	0.482	-0.810
3-day	3.608	0.475	-0.753
7-day	3.417	0.464	-0.666
15-day	3.240	0.461	-0.671
30-day	3.079	0.448	-0.668

Notes:

- Median plotting positions.
- Drainage area: 473 sq. miles.
- Period of systematic record: 1907-2010.
- Record length: 104 years.
- Regional skew values developed by USGS.
- Hollow points are censored events.

Figure 10. Unregulated frequency curves: Bellota

Regulated flow time series development

To develop regulated flow-frequency curves, the unregulated volume-duration-frequency curves are transformed through the unregulated-regulated flow transform. The unregulated-regulated flow transform captures the system's response to large, varied events, and is created using the unregulated and regulated flow time series. To develop the regulated flow time series we took selected historical events from the unregulated flow time series and simulated those in the regulated system. In addition, scaled historical events were used to represent events larger than those seen in the historical record for definition of the flow transforms. We then compiled the maximum unregulated and regulated flows for various durations to develop the event maxima datasets.

For this analysis we developed the regulated flow time series at each analysis location by:

- Smoothing the unregulated flow time series, using those series as boundary conditions to the reservoir simulation model.
- Identifying floods-of-record (discrete events) required to develop the flow transforms.
- Scaling historical events to represent events larger than those in the historical record.
- (Task 5.1 and Task 5.2) Simulating and routing reservoir releases of historical and scaled events.

Smooth unregulated flow time series

The daily unregulated flow time series are appropriate for frequency analysis. However daily upstream and intermediate boundary conditions do not have the temporal resolution required by the CVHS procedures for assessing the effects of regulation, particularly releases as indicated on the emergency spillway release diagram (ESRD). Therefore, the daily reservoir inflows and daily estimated local flows were "smoothed" to hourly time series. This smoothing was completed using a mass balance algorithm that interpolates the shape of the hydrograph and estimates peak hourly flows while maintaining daily volumes consistent with the original time series. These smoothed times series were provided by the Sacramento District Hydrology Section for use in this analysis.

Identify floods-of-record

Events rarer than p=0.5 annual exceedence are needed to define the flow transforms. To develop the flow transforms we used both historical events and scaled historical events. The 60 historical events used were those with 1-day volumes greater than 5,000 cfs (a threshold slightly lower than volume corresponding to the p=0.5 exceedence event.)

To select the subset of events used for scaling, we identified: (1) the 14 large flood events for the San Joaquin River basin (listed in the Comp Study historical storm matrices), and (2) the 5 largest events for the Calaveras River watershed. We list these events in Table 6. In Table 6, column 1 lists the water year of the event, column 2 and column 3 list the associated start and end dates, column 4 lists the 1-day volume, and column 5 indicates the selection basis. We identified these dates by visual inspection of unregulated inflow time series provided by the Corps. The time windows defined by these dates was used for extraction of the event maxima (unregulated and regulated) for development of the flow transforms.

The Comp Study lists both a January and February event for the 1969 water year in the San Joaquin River basin. However, a large February inflow event is not present in the New Hogan Reservoir unregulated inflow time series. Therefore, for this analysis we treat the 1969 flood as a single event.

Table 6. Calaveras River floods-of-record scaled to develop flow transforms

Water year[1] (1)	Start date (2)	End date (3)	1-day max volume (cfs) (4)	Selection basis (5)
1958	3/10/1958	4/30/1958	32,920	Largest inflow event
1938	1/25/1938	2/28/1938	30,450	Largest inflow event
1911	1/10/1911	2/28/1911	30,175	Largest inflow event
1936	2/10/1936	3/24/1936	26,987	Largest inflow event
1907	3/1/1907	4/14/1907	23,641	Largest inflow event
1986	2/10/1986	3/6/1986	23,494	Comp Study storm matrix event
1956	12/15/1955	2/15/1956	20,156	Comp Study storm matrix event
1998	1/1/1998	3/15/1998	16,919	Comp Study storm matrix event
1997	12/1/1996	2/15/1997	16,801	Comp Study storm matrix event
1969[2]	1/5/1969	3/20/1969	14,674	Comp Study storm matrix event
1940	2/11/1940	3/16/1940	13,610	Comp Study storm matrix event
1965	12/18/1964	1/18/1965	12,789	Comp Study storm matrix event
1982	12/28/1981	1/31/1982	12,321	Comp Study storm matrix event
1983	2/25/1983	4/10/1983	10,433	Comp Study storm matrix event
1995	3/1/1995	4/6/1995	10,146	Comp Study storm matrix event
1951	11/12/1950	11/31/1950	9,390	Comp Study storm matrix event
1980	1/1/1980	1/31/1980	8,648	Comp Study storm matrix event
1967	1/20/1967	2/10/1967	6,738	Comp Study storm matrix event
1978	3/1/1978	3/19/1978	5,770	Comp Study storm matrix event

1. Events are in order of increasing 1-day flow volume
2. For the purposes of this analysis we treat the 1969 flood as 1 single event.

Scale historical floods

In addition to the 60 historical floods-of-record, events larger than these recorded were required to develop the flow transforms throughout the full range of interest. To obtain those, we scaled the time series for the subset of historical events listed in Table 6 uniformly by factors at 0.2 intervals from 1.2 through 3.0 for use in simulating reservoir releases. This yielded a total of 10 scaled time series for each event. Both the unregulated reservoir inflow and estimated local flow time series were scaled uniformly to maintain the coincidence and timing of the system.

Scaled historical events were used only for the development of the flow transforms. The events were not used for fitting the unregulated flow frequency curves. This use of scaled historical events is consistent with the guidance in EM 1110-2-1415.

Simulate and route historical and scaled floods

We simulated reservoir operation and routed flows for both the historical floods-of-record and scaled historical events using the computer program HEC-ResSim, version 3.1 Beta III, developed by the USACE Hydrologic Engineering Center (HEC). Given a reservoir network, operating rules and constraints, and a set of inflows and downstream local flows, HEC-ResSim routes the flows through the system and simulates releases for the reservoirs. These releases are based on the rules and constraints defined in the water control manual.

An HEC-ResSim reservoir network includes representation of the physical properties of the reservoirs and links from reservoirs to downstream points of interest. Hydrologic routing model parameters are required to represent the movement of the flood wave between nodes in the network. Required physical properties include elevation-volume relationships, elevation-maximum outflow relationships, and physical limitations of the reservoir outlets.

The operating rules defined for a reservoir for HEC-ResSim include release functions based on reservoir pool elevation, reservoir inflow, and downstream flow constraints. Rate of change constraints are also included in the operation rule sets. For the Calaveras River, New Hogan Reservoir operates to meet downstream flow constraints at Bellota, which is the bifurcation of the Calaveras River and Mormon Slough approximately 18 miles downstream of the reservoir.

Simulate reservoir operation

For this analysis, we used the representation of the Calaveras River system in HEC-ResSim developed by the Corps; that will be used for the CVHS. This includes a representation of the network and the reservoir operation rules. The HEC-ResSim schematic of the Calaveras River system is shown in Figure 11.

For reference, New Hogan Reservoir is operated to maintain flows in the Calaveras River at Bellota below 12,500 cfs. The complete set of operating rules is defined in the New Hogan Reservoir water control manual (USACE 1983).

With this model, we simulated the 19 historical floods-of-record and associated scaled events for a total of 209 simulations. Consistent with the standard-of-practice for such analysis, for the reservoir routings, we used only the dedicated flood control storage space for the attenuation of the reservoir inflows. Thus, at the start of the simulation, the reservoir water surface elevation equals the elevation of the bottom of the flood control pool. The simulation time step for this analysis is 1 hour.

After completing the reservoir simulations, we reviewed the results from the HEC-ResSim computer program. Based on our knowledge of the system operation and water control manual, we reviewed and adjusted the HEC-ResSim computed flows. In several cases, we modified the reservoir releases using both release overrides and HEC-DSS routing computations to fully utilize the downstream channel capacity and available flood storage in the reservoir.

Route reservoir releases

We used Muskingum routing to route flows on the Calaveras River. A detailed channel model of the Calaveras River does not currently exist. Although the *Procedures document* calls for the hydraulic routing of reservoir releases, we found that the Calaveras River can be adequately simulated with hydrologic routing because: (1) the analysis locations on the Calaveras River are not affected by backwater and therefore do not require evaluation of stages to develop regulated flow-frequency curves, and (2) the reservoir release hydrographs do not rise quickly.

We reviewed the reservoir simulations and routings computed the program HEC-ResSim and adjusted as needed to obtain accurate peak regulated flows for the simulation of each event.

The results from the reservoir simulation and routing are provided on DVD with the original report.

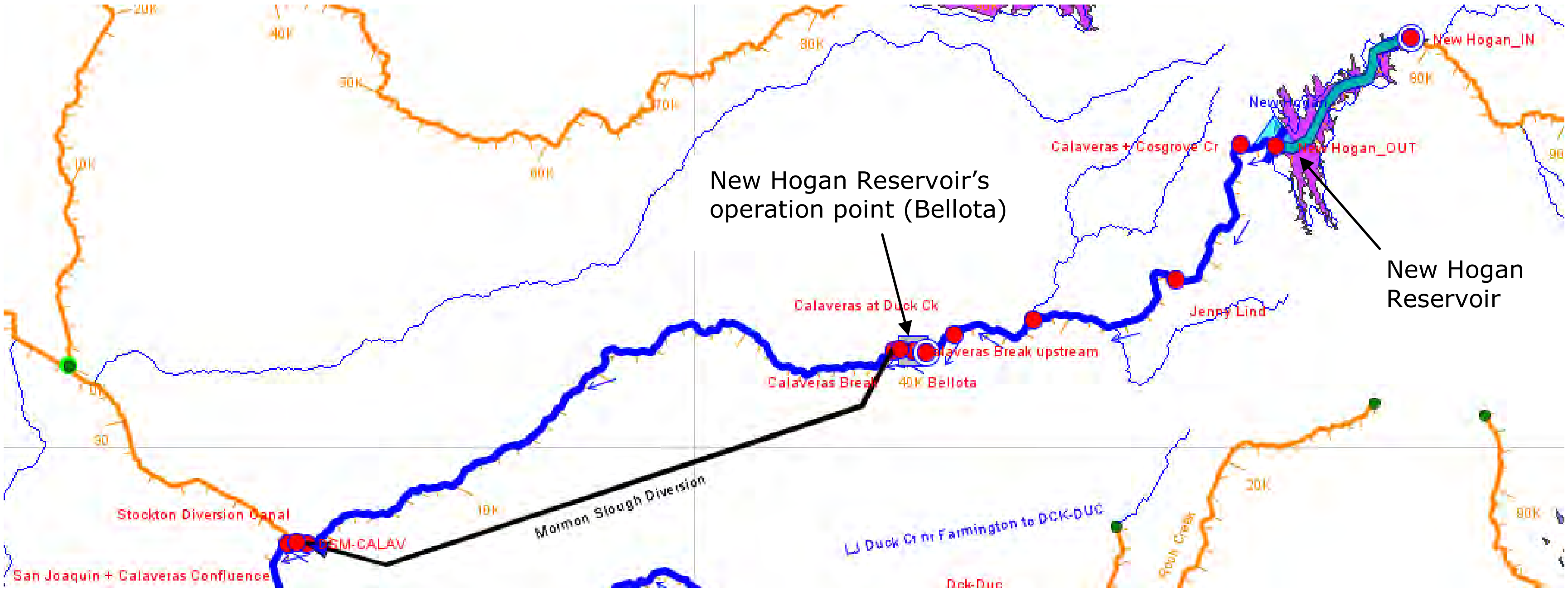


Figure 11. Screenshot of HEC-ResSim system schematic: Calaveras system

Flow transform fitting and application

Once the regulated flow time series were developed, the next step was to pair, by event, the unregulated and regulated flow time series. Using these pairings, the event properties, such as the volumes for given durations, and in the case of the regulated time series, peak flows, were identified. The result of this pairing and identification was the event maxima dataset. Specifically, the event maxima dataset consists of unregulated and regulated flows of various durations for a given historical or scaled historical event.

Once the event maxima datasets were compiled, a transform curve was fitted to develop the unregulated-regulated flow transforms. This curve translated the unregulated flow of a given quantile to the corresponding regulated flow for that same quantile. This process is illustrated in Figure 12.

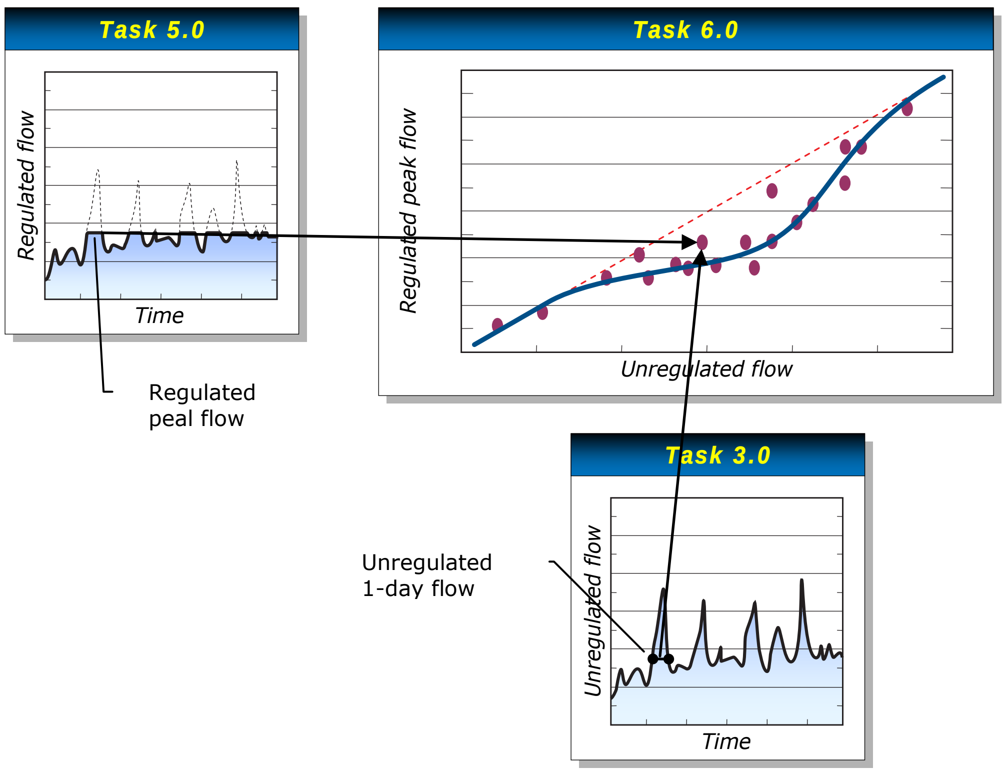


Figure 12. Flow transform development process

For the unregulated-regulated flow transform, the regulated flow value used was the peak flow. The unregulated flow value was the unregulated flow corresponding to the critical duration for that analysis location. The critical duration was found through an analysis of unregulated and regulated flows for historical and scaled historical events.

Additional transform curves were fitted to develop the family of characteristic curves. These curves identified the associated regulated volume duration characteristics of a given peak regulated flow.

For this analysis, we developed the flow transforms by:

- (Task 6.1) Identifying unregulated and regulated event maxima for the floods-of-record.
- (Task 6.2) Fitting the unregulated-regulated flow transform for each duration of interest.
- Determining the critical duration to identify the appropriate unregulated-regulated transform to use at each analysis location.
- Fitting the family of characteristic curves.
- Reviewing and accepting the flow transforms.

We then applied the flow transforms to the unregulated frequency curves to develop the regulated flow-frequency curves (Task 6.4).

Identify event maxima datasets

We identified the event maxima datasets using inspection and HEC-DSS utilities. For each analysis location, we:

- Identified the properties of the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations for unregulated flows associated with the floods-of-record. The durations we used are consistent with those specified in the *Technical procedures document* for analyzing critical duration.
- Identified the peak regulated flows from the regulated flow time series of the historical floods-of-record and scaled historical events. Note that here, peak regulated flow corresponds to the maximum hourly value regulated flow time series, and not a true instantaneous peak.
- Identified the properties of the 1-, 3-, 7-, 15-, and 30-day durations for regulated flows associated with the historical floods-of-record and scaled historical events. We did not include all the durations used in the critical duration analysis consistent with those specified in the *Technical procedures document* and the current standard-of-practice for flow-frequency analysis.

The event maxima datasets are tabulated in an MS Excel file on a DVD provided with the original report. The tabulated information lists each historical and scaled historical event used in this analysis and the associated volumes for the (1) unregulated flow volumes corresponding to the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations, and (2) regulated flow volumes corresponding to the peak, 1-, 3-, 7-, 15-, and 30-day durations.

Fit unregulated-regulated flow transforms

We developed the unregulated-regulated flow transforms for the 2 analysis locations by fitting transform curves through the pairs of event unregulated volumes and regulated peak flows. The unregulated volumes used were the average flows associated with the durations previously noted. We fitted these curves to the data pairs of historical and scaled events using the robust locally weighted scatterplot smoothing (LOWESS) regression technique. (The LOWESS procedure is detailed in the *Technical procedure document*.)

Here, we fitted these transforms for the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations. The event maxima datasets include both historical and scaled events to define the extreme end of the flow transform curves. Fitting of the transforms are detailed in Attachment 5.

The CVHS analysis procedure requires 1 single unregulated-regulated transform for statements of probability. To identify which duration is most appropriate, the critical duration for the given analysis location must be determined as described in the next subsection.

Determine critical duration

We determined critical duration at each analysis location by: (1) applying the unregulated-regulated flow transforms to the unregulated flow–frequency curves to develop hypothetical regulated flow-frequency curves, and (2) identifying the duration of the unregulated annual maximum series that consistently estimates the largest flow for each probability. In selecting the critical duration, we considered both the "goodness of fit" of each transform and which duration estimates the greater peak regulated flows. This procedure is described in more detail in Attachment 5.

From this analysis we determined that the critical duration at New Hogan Reservoir is 3.5 days and at Bellota is 1 day. Thus, the appropriate unregulated-regulated flow transforms used in this analysis were associated with these durations. The critical duration associated with the downstream operation point is shorter than that of the reservoir because of the effects of local flow.

After determining the critical duration associated with each analysis location, we reviewed and adjusted the unregulated-regulated flow transforms initially fitted with the LOWESS procedure as detailed in Attachment 5. We then adopted the flow transforms for New Hogan Reservoir and Bellota shown in Figure 13 and Figure 15. In Figure 13 and Figure 15, some scaled historical event maxima for more common events, i.e., annual exceedence probabilities greater than p=0.01, have regulated peaks exceeding the channel capacity (12,500 cfs) because of large local flows.

Fit family of regulated characteristic curves

We developed the families of regulated characteristic curves for New Hogan Reservoir and at Bellota by fitting most likely curves through the pairs of event regulated volumes as average flows and regulated peak flows, similar to the procedure we used to fit the unregulated-regulated transforms. The data pairs (from the event maxima datasets) we used include both historical and scaled events to define the extreme ends of the flow transform curve.

The family of regulated characteristic curves for New Hogan Reservoir and Bellota are shown in Figure 14 and *Figure* 16, and are detailed in Attachment 6. These curves associate regulated peak flows to regulated characteristic volumes. We fitted characteristic curves for the 1-, 3-, 7-, 15-, and 30-day durations. We compare these families of curves in Figure 17.

On the Calaveras River, the typical duration of releases from New Hogan Reservoir for events in the given range of interest is less than 15 days. Therefore we include the 15-day and 30-day characteristic curves here for completeness, and in keeping with the CVHS procedures.

For New Hogan Reservoir, the 1-day and 3-day regulated volume characteristic curves are almost the same for regulated peaks of approximately 14,000 cfs-22,00 cfs, as shown in Figure 14. This is expected for ranges of regulated peaks because large inflow volumes associated with the events will result in similar releases for the shorter durations while the reservoir is able to maintain control. Similarly, the characteristic curves at Bellota are the same for ranges of regulated peaks, as shown in *Figure* 16.

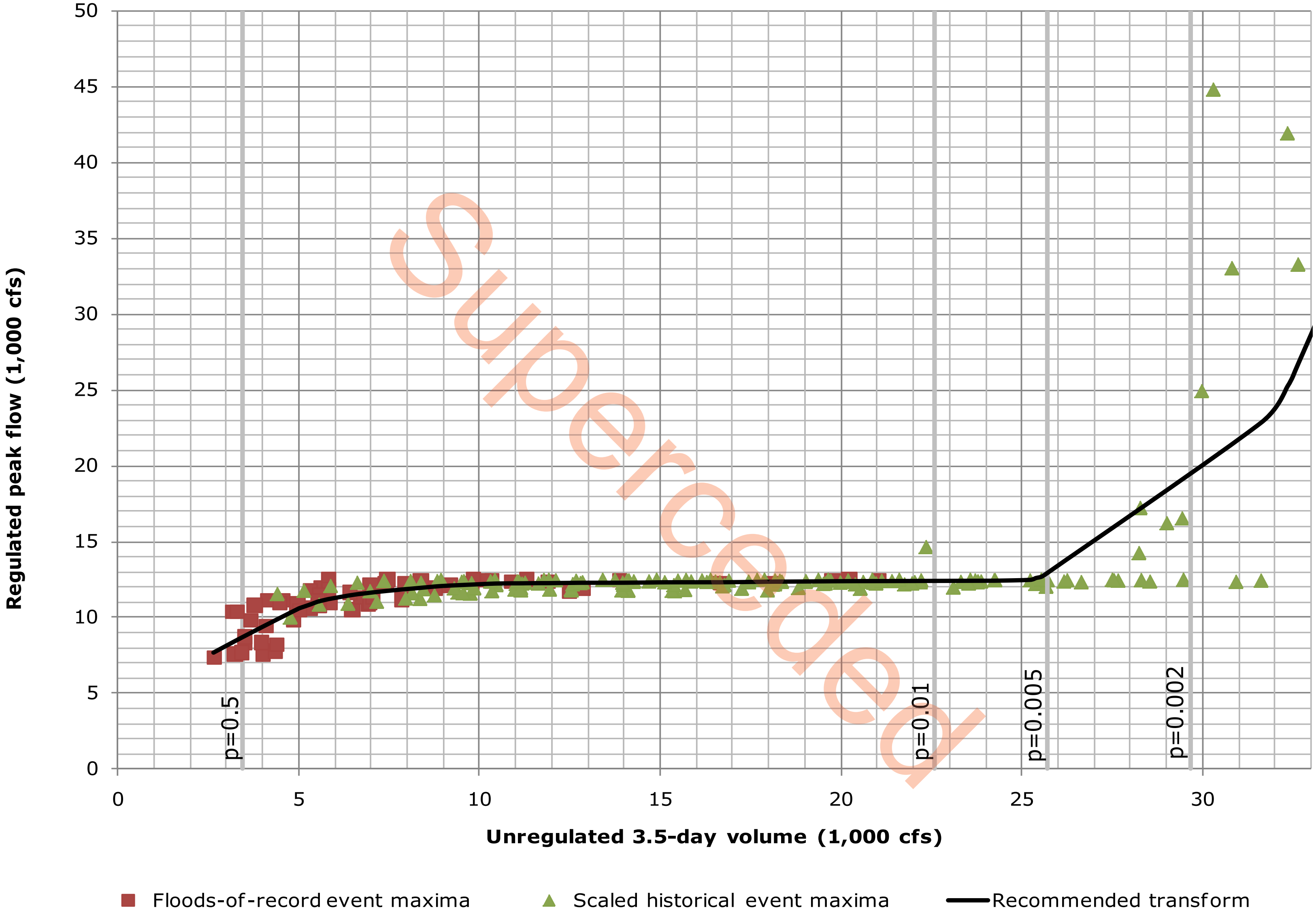


Figure 13. Unregulated-regulated flow transform: New Hogan Reservoir

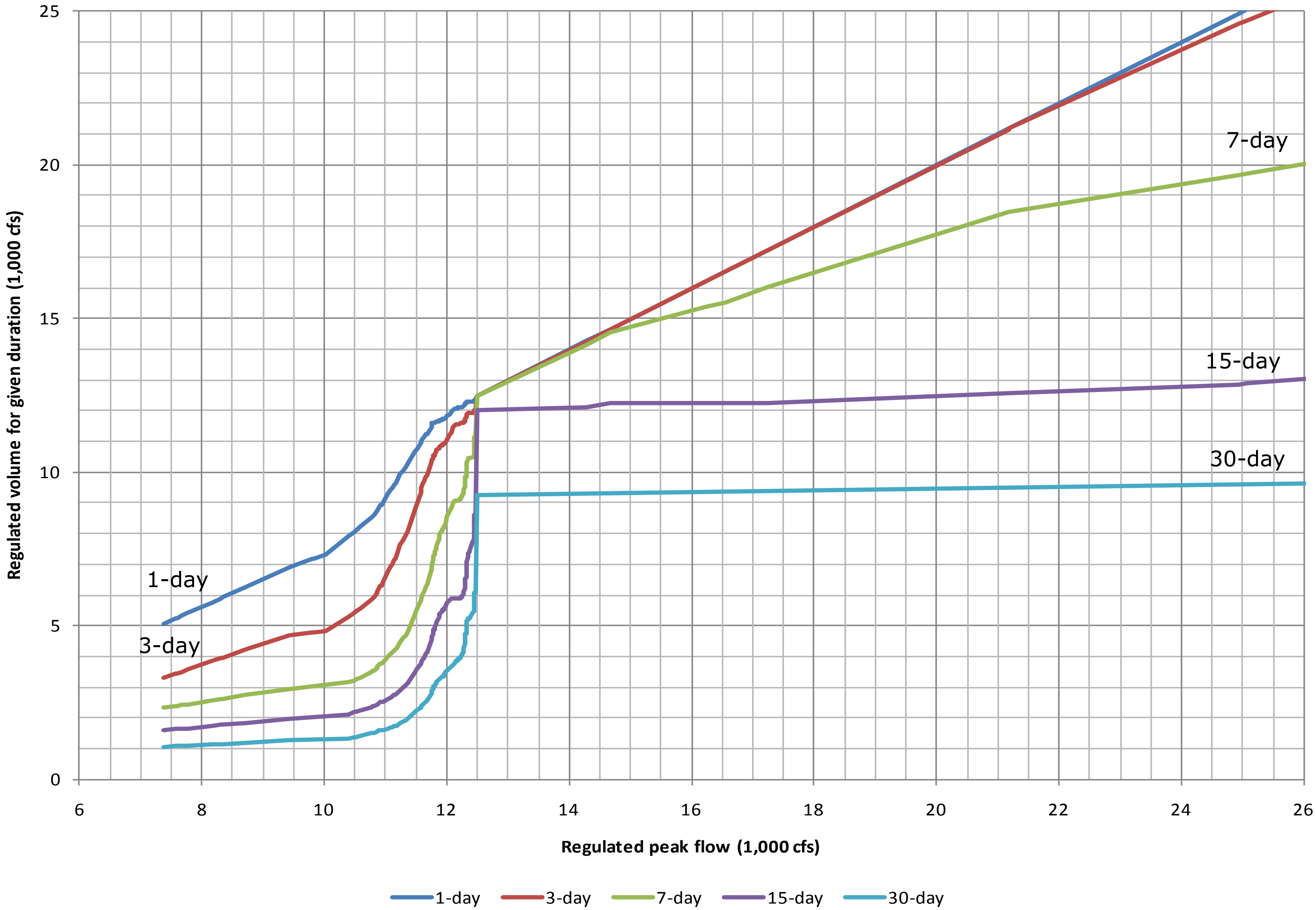


Figure 14. Family of regulated characteristic curves: New Hogan Reservoir

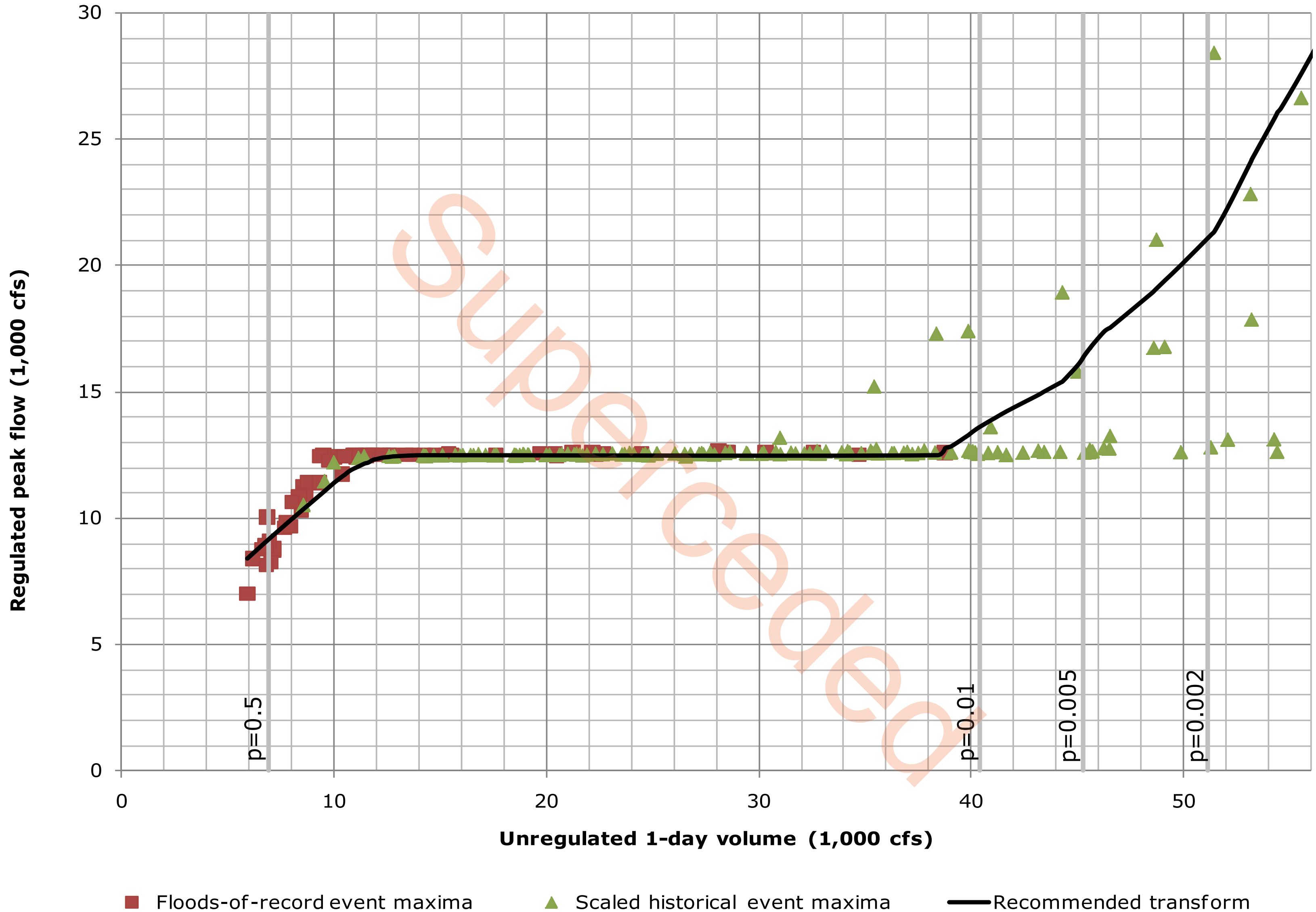


Figure 15. Unregulated-regulated flow transform: Calaveras River at Bellota

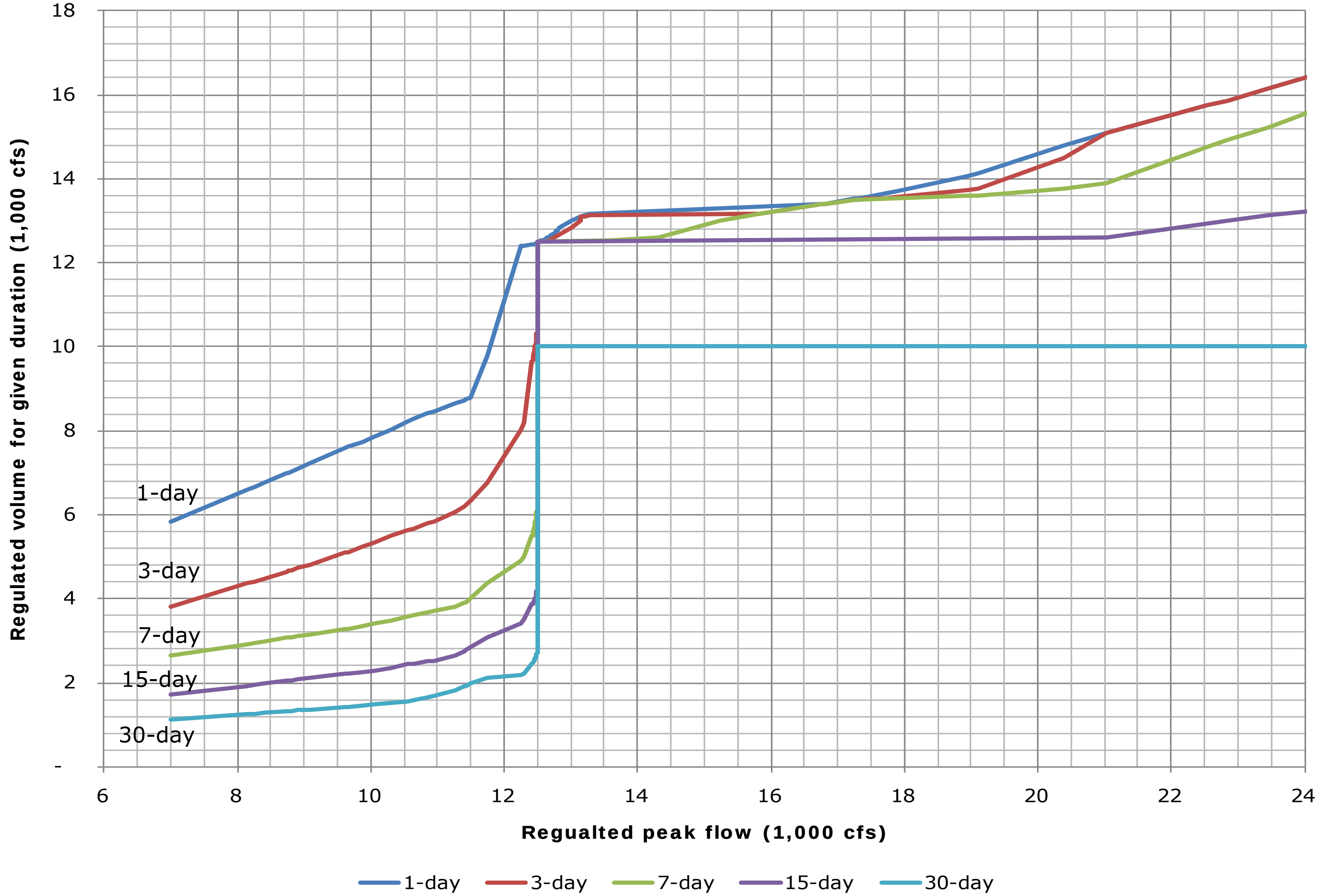


Figure 16. Family of regulated characteristic curves: Calaveras River at Bellota

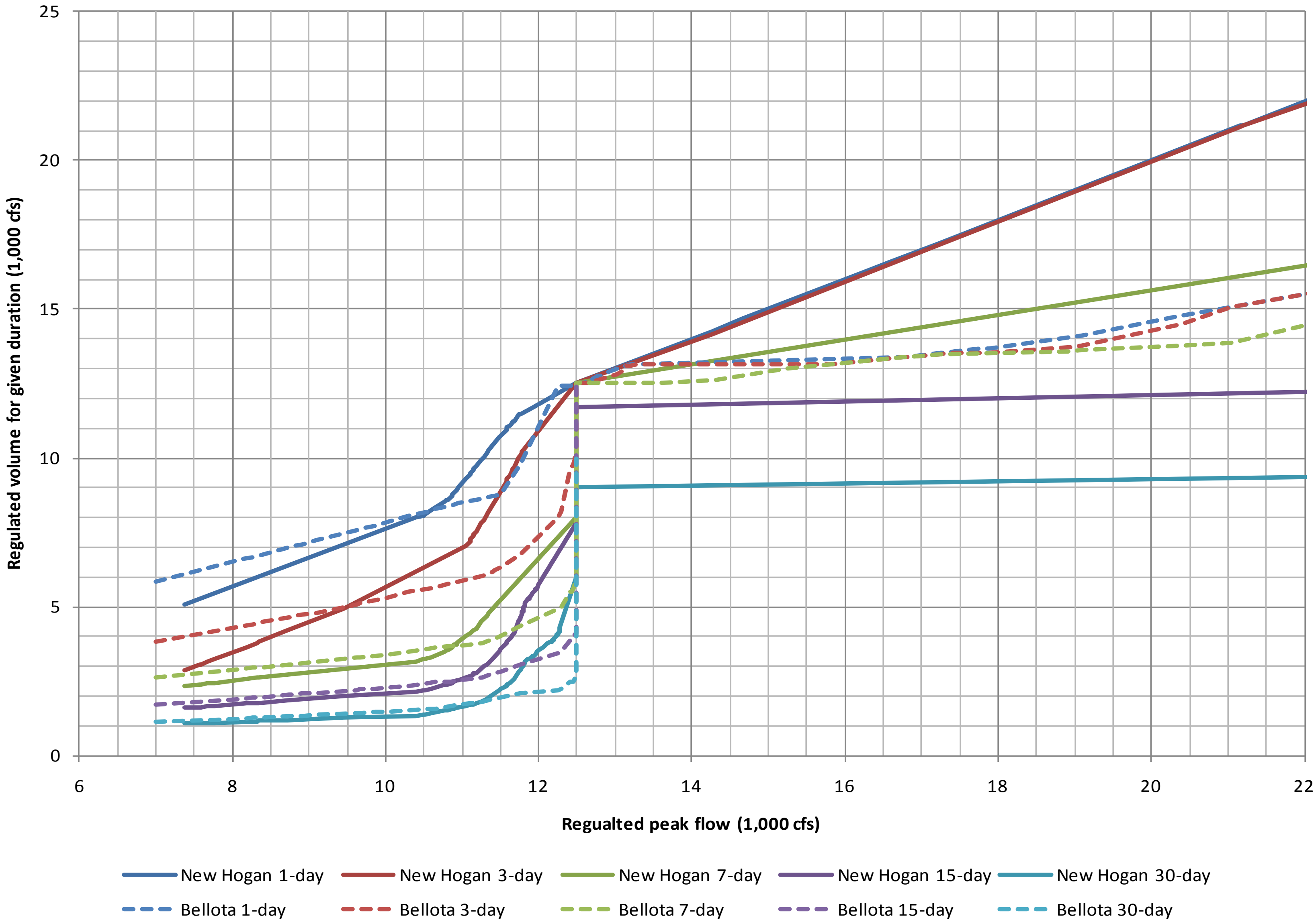


Figure 17. Comparison of the families of characteristic curves for New Hogan Reservoir and Bellota

Review and adopt flow transforms

After fitting the flow transforms and characteristic curves, we reviewed the resulting functions for consistency. Specifically, we compared each transform to (1) the transforms associated with different durations at the same analysis location, and (2) the transforms at the other analysis location. We found:

- The unregulated-regulated flow transforms were consistent between analysis location, i.e., the regulated peak flow for a given quantile at the downstream location was greater than that of the upstream location.
- At New Hogan Reservoir, the family of regulated characteristic curves were consistent between durations, i.e., they do not cross. This is expected.
- At Bellota, the initially fitted 3-day and 7-day curves crossed the 1-day curve. Therefore we set the 3-day characteristic curve equal the 1-day curve over their ranges of intersection, and the 7-day curve equal the 3-day curve over their initial range of intersection.
- The flow transforms at Bellota were sensitive to large peaks in local flow such as those computed directly for the 1997, 1998, and 2006 events. For scaled versions of these events, the local flow exceeded channel capacity before the New Hogan Reservoir flood control pool was filled.

Based on this review, we adopted these flow transforms for the 2 analysis locations.

Apply flow transforms

We developed a regulated peak flow-frequency curve and the associated regulated 1-, 3-, 7-, 15-, and 30-day volumes at New Hogan Reservoir and at Bellota by combining the appropriate information from the unregulated frequency curves, the flow transforms, and the families of regulated characteristic curves. The regulated flow-frequency curves for New Hogan Reservoir and Bellota are shown in Table 7 and Table 9 and their associated volumes are tabulated in Table 8 and Table 10.

To apply the flow transforms and develop regulated flow-frequency curve associated volumes at each analysis location we:

- Identified the unregulated flow quantiles associated with the critical duration that correspond to the probabilities of interest.
- Identified the regulated peak flows that correspond to the flow quantiles identified in the previous step using the flow transform.
- Identified the regulated flow characteristics that correspond to the regulated peaks identified in the previous step using the family of regulated characteristic curves.

Table 7. Regulated peak flow-frequency quantiles: New Hogan Reservoir

Annual exceedence probability (1)	1/annual exceedence probability (2)	Regulated peak flow (cfs) (3)
0.500	2	8,664
0.200	5	11,812
0.100	10	12,214
0.050	20	12,266
0.020	50	12,334
0.010	100	12,367
0.005	200	12,903
0.002	500	19,555

Table 8. Regulated peak flow values and associated volumes: New Hogan Reservoir

Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes[1] (as average flow for given duration)				
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)	30-day (cfs) (7)
0.500	8,664	6,212	4,188	2,720	1,843	1,199
0.200	11,812	11,625	10,634	7,457	4,994	3,096
0.100	12,214	12,107	11,582	9,098	5,909	3,963
0.050	12,266	12,140	11,607	9,312	6,032	4,157
0.020	12,334	12,283	11,880	10,275	7,045	5,120
0.010	12,367	12,300	11,916	10,459	7,411	5,263
0.005	12,903	12,900	12,893	12,876	12,026	9,283
0.002	19,555	19,555	19,549	17,462	12,445	9,463

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at New Hogan Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Table 9. Regulated peak flow-frequency quantiles: Calaveras River at Bellota

Annual exceedence probability (1)	1/annual exceedence probability (2)	Regulated peak flow (cfs) (3)
0.500	2	9,163
0.200	5	12,500
0.100	10	12,500
0.050	20	12,500
0.020	50	12,500
0.010	100	13,634
0.005	200	16,409
0.002	500	21,107

Table 10. Regulated peak flow values and associated volumes: Calaveras River at Bellota

Annual exceedence probability of regulated peak flow (1)	Regulated peak flow (cfs) (2)	Associated volumes[1] (as average flow for given duration)				
		1-day (cfs) (3)	3-day (cfs) (4)	7-day (cfs) (5)	15-day (cfs) (6)	30-day (cfs) (7)
0.500	9,163	7,271	4,852	3,163	2,127	1,372
0.200	12,500	12,500	12,500	12,500	12,500	10,000
0.100	12,500	12,500	12,500	12,500	12,500	10,000
0.050	12,500	12,500	12,500	12,500	12,500	10,000
0.020	12,500	12,500	12,500	12,500	12,500	10,000
0.010	13,634	13,174	13,141	12,545	12,515	10,001
0.005	16,409	13,367	13,300	13,300	12,553	10,002
0.002	21,107	15,106	15,106	13,930	12,631	10,005

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at New Hogan Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Expected hydrograph properties

The expected (design) hydrograph for a given exceedence probability is a New Hogan Reservoir outflow hydrograph with a peak flow that matched the regulated flow-frequency curve (as shown in Table 7) and with associated volumes matching those from the family of characteristic curves corresponding to the given regulated peak flow (as shown in Table 8). The properties of the expected hydrographs for the p=0.5, p=0.2, p=0.1, p=0.05, p=0.02, p=0.01, p=0.005, and the p=0.002 exceedence probabilities are shown in Table 11.

An expected hydrograph can be formed by applying these properties to a specific hydrograph shape. As part of future work, we will identify specific historical event patterns to which the expected hydrograph properties can be applied. For this identification, we will follow the example event selection procedure provided in the *CVHS Product uses* document (USACE 2009c) and .

Options for expected hydrograph development and application using study products were submitted by Ford Engineers to the Corps on June 23, 2010. From that memorandum, the Corps selection Option 1: Selected event-based reservoir release hydrographs.

Table 11. Expected hydrograph properties: New Hogan Reservoir outflow

Annual exceedence probability of regulated peak flow (1)	1/annual exceedence probability of peak flow (2)	Regulated peak flow (cfs) (3)	Associated volumes[1] (as average flow for given duration)				
			1-day (cfs) (4)	3-day (cfs) (5)	7-day (cfs) (6)	15-day (cfs) (7)	30-day (cfs) (8)
0.500	2	8,664	6,212	4,188	2,720	1,843	1,199
0.200	5	11,812	11,625	10,634	7,457	4,994	3,096
0.100	10	12,214	12,107	11,582	9,098	5,909	3,963
0.050	20	12,266	12,140	11,607	9,312	6,032	4,157
0.020	50	12,334	12,283	11,880	10,275	7,045	5,120
0.010	100	12,367	12,300	11,916	10,459	7,411	5,263
0.005	200	12,903	12,900	12,893	12,876	12,026	9,283
0.002	500	19,555	19,555	19,549	17,462	12,445	9,463

Notes:

1. These volumes were identified using the peak flows of the regulated flow-frequency curve at New Hogan Reservoir and the associated flow transforms, i.e., the family of regulated characteristic curves. These values are not a statement of probability.

Results

The results of this frequency analysis include:

- Unregulated frequency curves for New Hogan Reservoir (as shown in Figure 9).
- Unregulated frequency curves for the Calaveras River at Bellota (as shown in Figure 10).
- Unregulated-regulated flow transform for New Hogan Reservoir (as shown in Figure 13).
- Regulated flow-frequency curve and associated volumes for New Hogan Reservoir (as shown in Table 7 and in Table 8).
- Unregulated-regulated flow transform for the Calaveras River at Bellota (as shown in Figure 15).
- Regulated flow-frequency curve and associated volumes for the Calaveras River at Bellota (as shown in Table 9 and in Table 10).
- Expected hydrograph properties for New Hogan Reservoir (as shown in Table 11).

In addition, these intermediate data are included with the original report on DVD:

- HEC-DSS time series of the floods-of-records.
- HEC-DSS time series of the scaled historical floods.
- HEC-DSS time series of developed local flows below New Hogan Reservoir (detailed in Attachment 2).
- The tabulated event maxima datasets for the 2 analysis sites.
- Simulated reservoir releases and routed flows from the HEC-ResSim reservoir simulation model.
- Tabulated unregulated-regulated flow transforms for the 2 analysis sites.
- Tabulated families of regulated characteristic curves for the 2 analysis sites.

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Attachment 1: Correspondence of procedural steps

Table 12 shows how the procedural steps in this document correspond to the steps in the *Procedures document* and the *Technical procedures document*.

Table 12. Correspondence of procedural steps for the LSJR FS, the CVHS "Procedures document," and the CVHS "Technical procedures document"

This step in the hydrologic analysis at New Hogan Reservoir… (1)	**Corresponds to this action in the *Procedures document*… (2)**	**And/or this action in the *Technical procedures document*… (3)**
Develop unregulated flow time series	Task 3.0	Attachment B: Unregulated flow time series development
• Estimate local flows	Task 3.2	• Application and distribution of local flows
• Route and complete unregulated flow time series at analysis locations	Task 3.3	• Procedures for routing flows through the system
Develop unregulated frequency curves	Task 4.0	Attachment D: Frequency analysis
Develop regulated flow time series	Task 5.0	Attachment C: Regulated time series development
• Identify floods-of-record • Scaling of historical reservoir inflows	Task 6.2	Attachment E: Development of flow and stage transforms • Determination of historical event scaling for extrapolating unregulated-regulated flow transform
• Simulation of reservoir releases for historical and scaled events	Task 5.1, Task 6.2	• Procedures for routing regulated flows through the system
Develop flow transforms	Task 6.0	Attachment E: Development of flow and stage transforms
• Identify annual maximum series	Task 6.1	—
• Assess reservoir critical duration	—	Attachment E: Development of flow and stage transforms • Identification of critical duration at analysis points Attachment F: Procedure for critical duration calculation

This step in the hydrologic analysis at New Hogan Reservoir… (1)	Corresponds to this action in the *Procedures document*… (2)	And/or this action in the *Technical procedures document*… (3)
• Fit unregulated-regulated flow transform • Fit family of regulated characteristic curves	Task 6.3	Attachment E: Development of flow and stage transforms • Procedure for fitting a "most likely" transform through the datasets
• Apply flow transforms to develop regulated-flow-frequency curves	Task 6.4	—
Develop expected hydrographs[1]	—	—

Notes:

1. Options for expected hydrograph development using study products were submitted by Ford Engineers to the Corps on June 23, 2010. From that memorandum, the Corps selection Option 1: Selected event-based reservoir release hydrographs.

Attachment 2: Calaveras River local flow development

Overview

For the Calaveras River, we estimated local flows for the area between New Hogan Reservoir and Bellota, shown in Figure 8. For this area, we used 3 options to estimate local flow:

- Option 1. Direct calculation of local flow.
- Option 2: Estimation of local flow as a function of observed flow on Cosgrove Creek. Note: the Corps currently estimates local flow as 3.2 times observed (gaged) flow at Cosgrove Creek near Valley Springs, CA.
- Option 3. Estimation of local flow as a function of New Hogan Reservoir inflow.

Option 1 is the most accurate option for local flow estimation. To determine which of the other 2 options for local flow estimation is more appropriate to use, we:

- Reviewed the streamgage and reservoir inflow data provided by the Corps. In Table 13 we list the streamgages that were used in estimating local flows on the Calaveras River. Column 1 lists the streamgage ID whose corresponding name is listed in column 2, column 3 lists the data type (e.g., daily or hourly), column 4 lists the applicable time period of the streamgage data, and column 5 lists notes on the data.
- Coordinated with Corps staff regarding streamgage data quality.
- Identified the data type (e.g., daily or hourly) of the provided data.
- Identified the overlapping time periods for each streamgage by time step.
- Estimated local flow by direct calculation (Option 1).
- Compared the directly calculated local flow time series to observed flows on Cosgrove Creek and New Hogan Reservoir inflows.
- Identified, for Option 2 and Option 3, alternative functions for estimating local flow including:
    - Direct multipliers based on ratios of peak flows for selected large events.
    - Direct multipliers based on drainage area ratios.
    - Linear functions determined by regression analysis.
    - Exponential functions determined by regression analysis.
    - Linear functions of logarithmic transforms of flow determined by regression.
- Estimated local flow time series using the possible functions identified.
- Estimated a local flow time series using the observed flow on Cosgrove Creek and the 3.2 multiplier used by the Corps.

- Compared the estimated local flow time series to the directly calculated local flow time series.
- Identified the function for each option that most reasonably estimates local flows.

Table 13. Streamgages reviewed for use in estimating local flows on the Calaveras River: data were provided by Corps on 6/22/2010 as part of the CVHS.

USGS or CDEC ID (1)	Streamgage name (2)	Data type (3)	Time period (water year) (4)	Notes (5)
—	New Hogan Reservoir unregulated inflow	Daily	1907-2010	Values computed by Corps. Data start January 1, 1907.
NHG	New Hogan Dam (reservoir outflow)	Daily	1963-2009	
		Hourly	1995-2009	Data start January 1, 1995.
MRS	Mormon Slough at Bellota (USACE gage)	Daily	1988-2010	No data reported for the 1994 and 1995 flood season. Some data values are missing. Streamgage data are influenced by regulation.
		Hourly	1996-2010	Some data values are missing.
11308900	Calaveras River below New Hogan Dam near Valley Springs, CA	Daily	1961-2009	Data start January 1, 1961. Streamgage data are influenced by regulation.
11309000	Cosgrove Creek near Valley Springs, CA	Daily	1930-1969	
			1991-2010	Data start January 1, 1991. No data reported for the 1994 and 1995 flood season. Some data values are missing.
11309500	Calaveras River at Jenny Lind, CA	Daily	1907-1966	Data start January 1, 1907. Some data values are missing, particularly in the summer months. Streamgage data are influenced by regulation.
11310500	Calaveras River near Stockton, CA	Daily	1926	Data for 1 major flood event only. Streamgage data are influenced by regulation.
			1944-1950	Data for 1 major flood event only for each water year. Streamgage data are influenced by regulation.
			1976-1986	Some data values are missing. Streamgage data are influenced by regulation.

Event selection for local flow estimation analysis

As previously noted, local flows developed were used to support the development of an unregulated-regulated flow transform and a family of regulated characteristic curves. A key aspect in the development of these was the scaling of the largest events, i.e., the 19 events previously indentified for the Calaveras River.

Thus, the local flows estimated for these large events needed to be reasonable and as accurate as possible. To assess this, we used the local flows calculated directly corresponding to the largest events possible as a basis of comparison. Specifically, we used the 1997, 1998, and 2006 water year events whenever possible. Although the 2006 event is not included in 19 events previously indentified (because it is the 10th largest event on record on the Calaveras River and occurred after the completion of the Comp Study), it was useful in developing local flows. We defined the 2006 water year event as starting on 3/24/2006 and ending on 4/30/2006.

Local flow estimation Option 1: Calculate local flows directly

The preferred option for estimating local flows was to calculate directly flows using streamgage data. In general, this was completed on the Calaveras River using known releases from New Hogan Reservoir and the observed flows at Bellota. This was completed only for the time periods when data overlap.

In the case of daily data, local flows were calculated directly by subtracting the reservoir releases from the gaged flows. Any resulting negative values were then set to 0. Routing of the daily observed outflows (using the 1-hour hydrologic routing model of the Calaveras River) was not necessary because the total travel time between New Hogan Reservoir and Bellota is less than 1-day.

Accepted travel time estimates between New Hogan Reservoir and Bellota are: (1) 3 hours as indicated in the New Hogan Reservoir water control manual (Corps 1983), and (2) 7.05 hours as indicated by the sum of Muskigum K values from the HEC-ResSim model provided by the Corps. This longer travel time was attributed to the availability of hourly streamgage data after 1995 used to calibrate the reservoir simulation and hydrologic routing model of the Calaveras River, and was adopted for this analysis.

In the case of hourly data, reservoir releases were first routed from New Hogan downstream to the gage at Bellota. These routed releases were then subtracted from the observed flows to calculate local flow directly. Again, any resulting negative values are then set to 0. We used hydrologic routing to estimate local flows on the Calaveras River. Specifically, we used HEC-DSS math utilities and the Muskingum routing parameters from the CVHS HEC-ResSim model as shown in Table 14. In Table 14, column 2 lists the reach, column 3 lists the Muskingum K values in hours, column 4 lists the Muskingum X, and column 5 the number of subreaches.

In Table 15 we summarize how local flows were calculated directly by time period and data type. In Table 15, column 2 lists the data type, column 3 the overlapping time period, and column 4 the components for calculating local flows.

In Figure 18 through Figure 20 we compared the daily and hourly inferred local flows for the 1997, 1998, and 2006 water year events. (These events are the 3 largest of the overlapping time period for which we could calculate both daily and hourly local flows.) In Figure 18 through Figure 20 the daily local flows are shown in red, the hourly local flows in blue, and the daily differences in their volumes (daily local flows minus hourly local flows) in green. From these comparisons we see (1) that the timing of the hourly and daily local flows are similar, and (2) the differences in volume appear to be greatest around the largest local flows associated with the event. These differences in volumes are small compared to the total volume of unregulated inflow to New Hogan Reservoir.

Table 14. Calaveras River Muskingum routing parameters between New Hogan Reservoir and Bellota

ID (1)	Reach (2)	Muskingum K (hours) (3)	Muskingum X (4)	Number of subreaches (5)
1	New Hogan Reservoir to Cosgrove Creek[1]	—	—	—
2	Cosgrove Creek to Jenny Lind	1.05	0.2	1
3	Jenny Lind to Indian Creek	2.5	0.2	2
4	Indian Creek to Duck Creek	2.0	0.2	2
5	Duck Creek to Bellota	1.5	0.2	2
6	**Total**	**7.05**	**—**	**—**

Notes:

1. There was no routing for this reach.

Table 15. Summary of direct calculation of local flows on the Calaveras River

ID (1)	Data type (2)	Overlapping time period[1] (water year) (3)	Calculate local flows directly by:[2] (4)
1	Daily	1988-2009	Subtracting known outflows from New Hogan Reservoir from observed flows at Bellota
2	Hourly	1996-2009	Routing known outflows from New Hogan Reservoir, then subtracting these routed flows from observed flows at Bellota

Notes:

1. Because of missing values, local flow may not be calculated directly for the entire period listed. In such cases flows are either interpolated using the directly calculated flow, or Option 2 or Option 3 depending on data availability.
2. Any resultant negative values were set to 0.

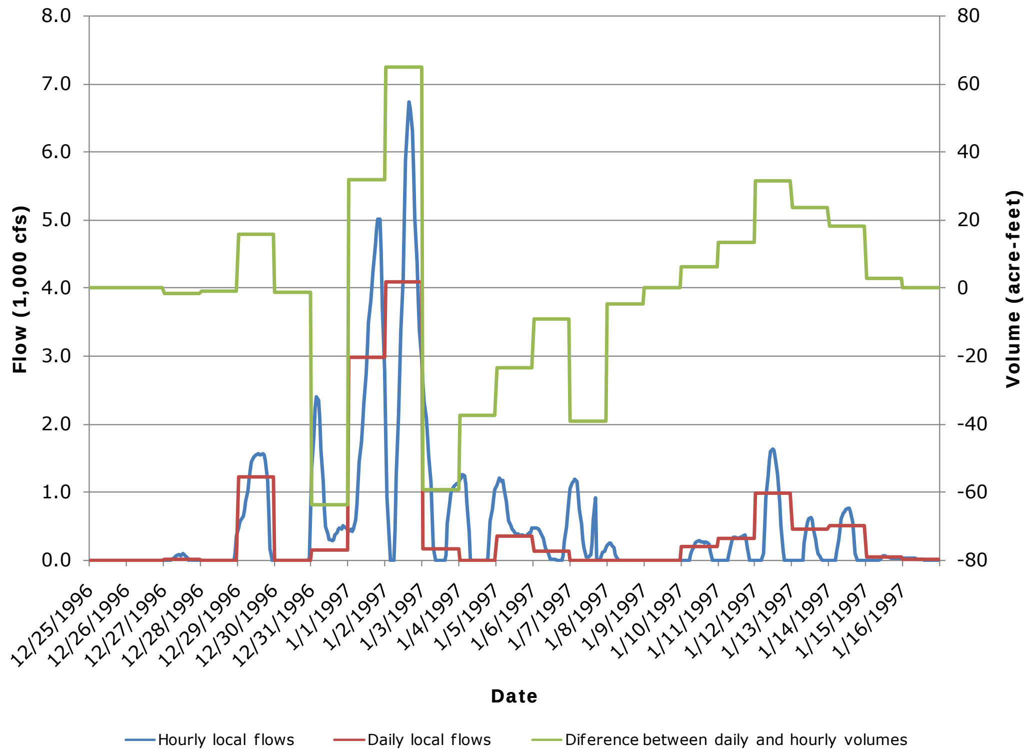

Figure 18. Calaveras River 1997 event directly calculated local flows

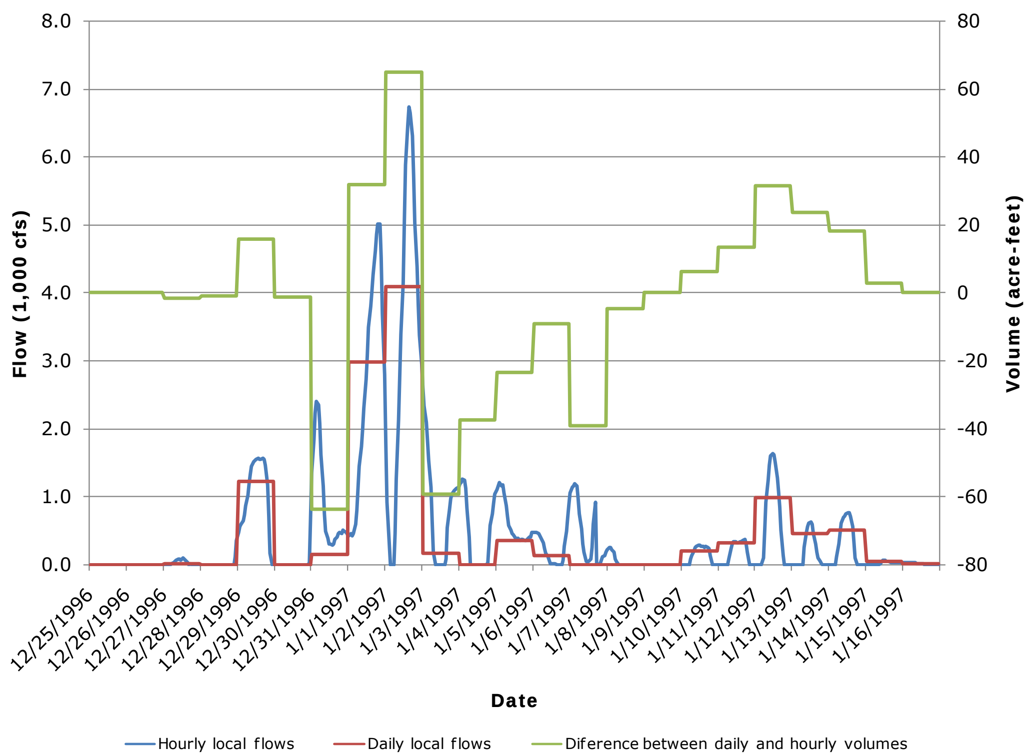

Figure 19. Calaveras River 1998 event directly calculated local flows

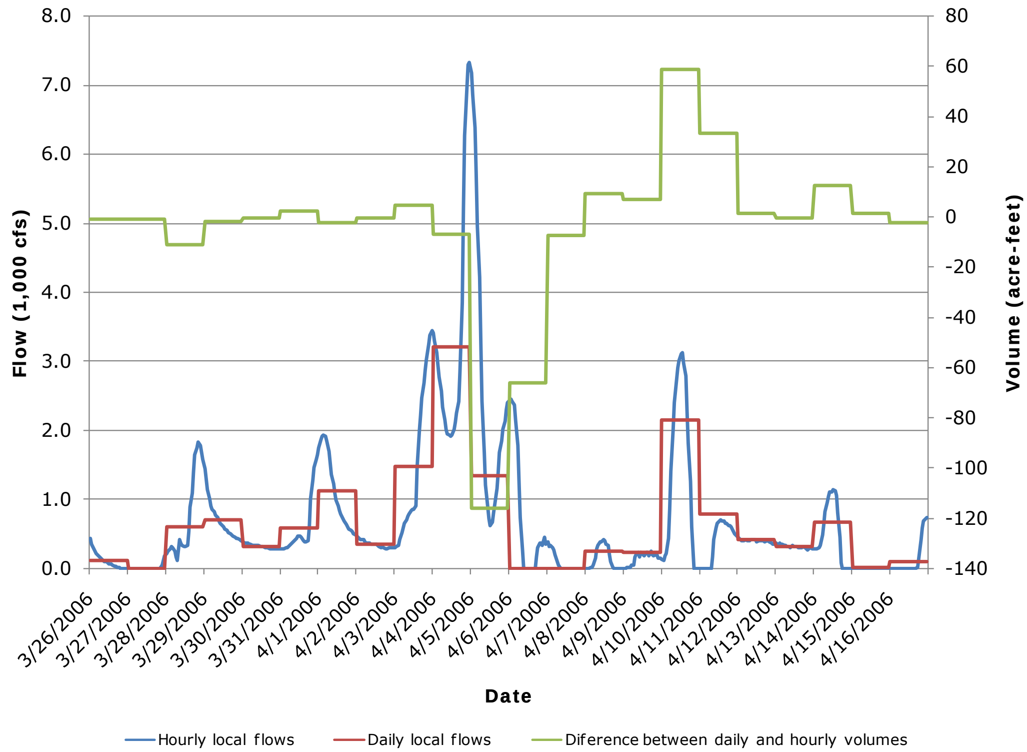

Figure 20. Calaveras River 2006 event directly calculated local flows

Local flow estimation Option 2: Estimate local flows as a function of observed flows of Cosgrove Creek

In the cases where local flows could not be calculated directly, we estimated local flows using nearby streamgages. As noted above, the Corps already estimates local flows using coefficients for reservoir operations on the Calaveras River as 3.2 times the observed flow at the Cosgrove Creek near Valley Springs, CA, streamgage. Because the estimation of local flows is important to simulate accurately reservoir operations we need to either (1) verify the coefficients used by the Corps to estimate such flows, or (2) adopt new coefficients. We completed this task by:

- Calculating local flows directly as detailed in the previous subsection.
- Comparing the directly calculated local flow time series to observed flows on Cosgrove Creek for selected large events occurring in the overlapping period of record.
- Identifying an average ratio of maximum 1-day flows on Cosgrove Creek to directly calculated peak local flows for selected large events.
- Estimating local flow time series using the average ratio identified as a multiplier of unregulated reservoir inflow.
- Estimating local flow time series using a drainage area ratio between the local flow area and Cosgrove Creek watershed as a multiplier to observed flows on Cosgrove Creek.

- Completing regression analyses that relate the directly calculated local flows to the observed flows on Cosgrove Creek for the overlapping periods of record. (Note that the Bear Creek near Lockeford, CA streamgage was also considered for regression analysis, however none of the record overlaps the period for which we can infer local flows directly and therefore the data were not used.)
- Identifying the best fitted functions from the regression analysis for estimation of local flows.
- Estimating local flow time series using the identified functions.
- Estimating a local flow time series using the observed flow on Cosgrove Creek and the 3.2 multiplier used by the Corps.
- Comparing the estimated local flow time series to the directly calculated local flow time series.
- Identifying the function that most reasonably estimates local flows.

Based on this analysis, we identified the best relation for estimating local flows using observed flow on Cosgrove Creek to be the function currently used by the Corps. Thus, we estimated local flows as:

$$Q_{Local} = 3.2\left(Q_{Cosgrove}\right) \tag{3}$$

where Q_{Local} is the local flow estimate for a given time, and $Q_{Cosgrove}$ is the observed flow at the Cosgrove Creek near Valley Springs, CA, streamgage for that same time. All estimated local flows using this option were on a daily basis. We did not lag or route the estimated flows because: (1) synthesizing a shorter time step is not required for frequency analysis, and (2) the travel time between the Cosgrove Creek gage and Bellota is approximately 7 hours, which is less than the 1-day time step of the observed flows.

Local flow estimation Option 3: Estimate local flows as a function of unregulated inflow to New Hogan Reservoir

In the cases where local flows could not be inferred directly or estimated using nearby streamgages, we estimated local flows using reservoir inflows. We determined the function that most reasonably estimates local flow using the same procedure previously detailed for estimating flows as a function of observed flows on Cosgrove Creek.

Based on this analysis, we identified the best function for estimating local flows using unregulated inflows to New Hogan Reservoir as:

$$Q_{Local} = 0.226\left(Q_{NHG}\right) \tag{4}$$

where Q_{Local} is the local flow estimate for a given time, and Q_{NHG} is the unregulated inflow to New Hogan Reservoir. All estimated local flows using this option were on a daily basis. We did not lag or route the estimated flows because: (1) synthesizing a shorter time step is not required for frequency analysis, and (2) the travel time between the Cosgrove Creek gage and Bellota is approximately 7 hours, which is less than the 1-day time step of the inflows.

Local flow estimation details

The selected estimation approaches, in order of best estimate of local flow, are:

- Option 1. Calculate local flow directly using known releases from New Hogan Reservoir and the observed flows at Bellota, routing hourly flows as necessary. Note in the case of missing streamgage data, local flows values were interpolated as needed.
- Option 2. Estimate local flow as 3.2 times the observed flow at the Cosgrove Creek near Valley Springs, CA, streamgage.
- Option 3. Estimate local flow as 0.226 times the unregulated inflow to New Hogan Reservoir.

We detail the development of the local flow time series for New Hogan Reservoir in Table 16. Column 1 notes the time period for which the option listed in column 3 will be used to estimate local flow, and column 2 lists the time step (hourly or daily) of the developed local flow time series. We interpolated local flows using other estimated local flows as appropriate. The hourly and daily time series were combined and these finalized time series stored as hourly data in HEC-DSS.

Table 16. Local flow time series calculation details by time period

Time period (date) (1)	Time step (2)	Approach to be used (3)
1/1/1907-9/30/1929	Daily	Option 3: 0.226 times reservoir inflow.
10/1/1929-9/30/1969	Daily	Option 2: 3.2 times Cosgrove Creek flow.
10/1/1969-12/31/1987	Daily	Option 3: 0.226 times reservoir inflow.
1/1/1988-9/19/1988	Daily	Option 1: directly infer local flow.
9/20/1988-3/25/1989	Daily	Option 3: 0.226 times reservoir inflow.
3/26/1989-3/29/1989	Daily	Option 1: directly infer local flow.
3/30/1989-5/1/1989	Daily	Option 3: 0.226 times reservoir inflow.
5/2/1989-8/13/1989	Daily	Option 1: directly infer local flow.
8/14/1989-1/3/1990	Daily	Option 3: 0.226 times reservoir inflow.
1/4/1990-2/27/1991	Daily	Option 1: directly infer local flow.
2/28/1991-3/6/1991	Daily	Option 2: 3.2 times Cosgrove Creek flow.
3/7/1991	Daily	Option 1: directly infer local flow
3/8/1991-3/11/1991	Daily	Option 2: 3.2 times Cosgrove Creek flow.
3/12/1991-3/25/1991	Daily	Option 1: directly infer local flow.
3/27/1991-9/30/1991	Daily	Option 1: directly infer local flow.
10/1/1991-12/31/1991	Daily	Option 2: 3.2 times Cosgrove Creek flow.
1/1/1992-11/1/1993	Daily	Option 1: directly infer local flow.
11/2/1993-6/1/1995	Daily	Option 3: 0.226 times reservoir inflow.
6/2/1995-10/20/1995	Daily	Option 2: 3.2 times Cosgrove Creek flow.
10/21/1995-12/15/1995	Hourly	Option 1: directly infer local flow.
12/16/1995-12/20/1995	Daily	Option 2: 3.2 times Cosgrove Creek flow.
12/21/1995	Hourly	Option 1: directly infer local flow.
12/22/1995	Daily	Option 3: 0.226 times reservoir inflow.
12/23/1995	Daily	Option 2: 3.2 times Cosgrove Creek flow.
12/24/1995-12/25/1995	Hourly	Option 1: directly infer local flow.
12/26/1995-1/2/1996	Daily	Option 2: 3.2 times Cosgrove Creek flow.
1/3/1996-8/13/2009	Hourly	Option 1: directly infer local flow.
8/14/2009-3/14/2010	Daily	Option 2: 3.2 times Cosgrove Creek flow.
3/15/2010-7/8/2010	Daily	Option 3: 0.226 times reservoir inflow.

Attachment 3: Annual maximum series for unregulated frequency curves

Here we list the series of annual maximum unregulated volume values that we used in development of the unregulated frequency curves for New Hogan Reservoir and at Bellota. In addition, we include here the unregulated peak inflow annual maximum series for New Hogan Reservoir. Development of a peak flow-frequency curves is not required for development of the regulated flow-frequency curves. However, we developed such curves for completeness.

Annual maximum series

For the New Hogan Reservoir, the unregulated reservoir inflow time series was used as the basis of the unregulated frequency analysis. The Corps provided the finalized unregulated inflow time series for New Hogan Reservoir on 7/12/2010. From this time series, we extracted the 1-, 3-, 7-, 15-, and 30-day volume data. We list these values for New Hogan Reservoir in Table 17. In the table, column 1 lists the water year, and columns 2 through 11 list the date, if available, and the volume, as average flow for the given duration, in cfs. The dates listed in Table 17 correspond to the start of the duration.

To develop annual maximum series for New Hogan Reservoir's operation point on the Calaveras River at Bellota, we combined the unregulated inflow time series with the estimated local flows by adding the 2 time series together using HEC-DSS math utilities. Note that we did not route the unregulated reservoir inflows because the travel time between the reservoir and the operation point is less than the time step of the inflows: 1 day.

Using these data, we computed the 1-, 3-, 7-, 15-, and 30-day volume-duration data using HEC-SSP version 1.1. We list these values for Bellota in Table 18. In the table, column 1 lists the water year, and columns 2 through 11 list the date, if available, and the volume, as average flow for the given duration, in cfs. The dates listed in Table 18 correspond to the start of the duration.

In addition, we reviewed the computed values for consistency. Specifically, we checked that the extracted value for a given duration is less than the values associated with each shorter duration in a given water year. For both analysis locations, we found that the computed values for each water year decrease as duration increases.

Table 17. New Hogan Reservoir annual maximum series for unregulated volume-frequency analysis

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1907	3/19/1907	23,641	3/19/1907	13,508	3/23/1907	9,285	3/24/1907	7,065	4/2/1907	4,550
1908	2/10/1908	2,028	2/11/1908	1,122	2/14/1908	620	1/28/1908	473	2/12/1908	429
1909	1/21/1909	17,875	1/22/1909	8,188	1/26/1909	5,176	1/27/1909	4,474	2/12/1909	3,374
1910	12/9/1909	7,150	12/9/1909	3,344	12/11/1909	2,098	12/15/1909	1,463	1/3/1910	919
1911	1/31/1911	30,175	2/1/1911	20,489	1/31/1911	10,686	2/3/1911	6,714	2/10/1911	4,402
1912	3/13/1912	1,076	3/15/1912	642	3/19/1912	480	3/20/1912	369	4/4/1912	249
1913	1/19/1913	1,278	1/19/1913	779	1/21/1913	557	1/29/1913	345	2/13/1913	202
1914	2/21/1914	8,745	2/21/1914	6,179	1/28/1914	3,972	1/28/1914	2,793	1/29/1914	1,926
1915	2/1/1915	8,092	2/3/1915	6,922	2/3/1915	4,480	2/11/1915	3,610	2/26/1915	2,320
1916	3/20/1916	9,543	3/22/1916	4,520	1/30/1916	2,978	1/28/1916	2,594	2/7/1916	2,197
1917	2/21/1917	18,932	2/23/1917	13,742	2/27/1917	8,302	3/6/1917	4,631	3/20/1917	2,729
1918	3/11/1918	16,241	3/12/1918	11,737	3/13/1918	6,641	3/21/1918	3,859	3/24/1918	2,279
1919	2/11/1919	7,150	2/12/1919	3,802	2/16/1919	1,844	2/24/1919	1,022	3/11/1919	849
1920	3/17/1920	2,854	3/23/1920	2,386	3/22/1920	1,908	3/24/1920	1,263	3/30/1920	835
1921	1/18/1921	23,641	1/20/1921	10,943	1/23/1921	5,251	1/31/1921	3,267	2/15/1921	1,951
1922	2/20/1922	9,024	2/11/1922	7,608	2/14/1922	3,873	2/23/1922	3,068	3/9/1922	1,804
1923	12/13/1922	6,756	12/14/1922	5,234	12/16/1922	2,931	12/21/1922	1,632	1/5/1923	1,093
1924	2/6/1924	173	2/8/1924	162	2/12/1924	105	2/15/1924	81	2/15/1924	61
1925	2/6/1925	12,685	2/7/1925	6,333	2/10/1925	3,296	2/18/1925	2,073	3/5/1925	1,370
1926	2/14/1926	2,941	2/14/1926	2,508	2/18/1926	1,494	2/17/1926	978	2/28/1926	642
1927	2/4/1927	5,747	2/5/1927	3,495	2/21/1927	2,571	2/18/1927	1,658	3/4/1927	1,355
1928	3/25/1928	10,283	3/26/1928	6,490	3/30/1928	4,187	4/7/1928	2,371	4/22/1928	1,314
1929	2/4/1929	1,557	2/5/1929	980	2/8/1929	578	2/15/1929	325	2/17/1929	218
1930	3/6/1930	3,460	3/7/1930	3,053	3/10/1930	1,758	3/9/1930	1,151	3/24/1930	714

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1931	2/15/1931	866	2/17/1931	492	2/21/1931	380	2/28/1931	244	3/15/1931	171
1932	12/28/1931	11,600	2/9/1932	8,430	2/11/1932	5,501	2/14/1932	3,905	2/28/1932	2,177
1933	1/28/1933	1,866	1/30/1933	1,688	1/30/1933	1,262	2/3/1933	751	2/18/1933	509
1934	1/2/1934	5,262	1/2/1934	3,556	1/4/1934	2,490	3/5/1934	1,364	3/9/1934	831
1935	3/8/1935	7,270	4/10/1935	5,745	4/10/1935	4,065	4/18/1935	2,941	5/2/1935	1,893
1936	2/23/1936	26,987	2/24/1936	21,856	2/26/1936	12,506	2/26/1936	11,470	3/2/1936	6,484
1937	2/6/1937	17,805	2/7/1937	15,114	2/10/1937	7,987	2/16/1937	5,462	2/27/1937	3,490
1938	2/11/1938	30,450	2/13/1938	20,914	2/16/1938	13,451	2/15/1938	9,114	3/4/1938	5,637
1939	2/8/1939	2,387	2/9/1939	1,281	2/13/1939	751	2/14/1939	506	3/1/1939	350
1940	3/4/1940	13,610	2/29/1940	10,597	3/4/1940	8,262	3/8/1940	4,750	3/4/1940	2,800
1941	4/4/1941	9,036	3/3/1941	6,660	3/6/1941	4,742	3/8/1941	2,983	3/9/1941	2,629
1942	1/27/1942	15,522	1/28/1942	11,557	1/30/1942	8,104	2/7/1942	5,287	2/21/1942	3,128
1943	1/21/1943	12,420	1/23/1943	9,336	3/11/1943	8,229	3/19/1943	5,619	3/26/1943	3,825
1944	2/3/1944	6,498	2/5/1944	4,471	2/8/1944	2,608	2/16/1944	1,617	3/2/1944	1,021
1945	12/23/1944	4,221	12/24/1944	3,351	12/28/1944	2,757	1/5/1945	1,881	1/19/1945	1,185
1946	3/10/1946	1,295	3/12/1946	980	3/16/1946	654	3/18/1946	448	4/8/1946	403
1947	3/25/1947	1,557	4/8/1947	1,071	4/25/1947	946	5/2/1947	890	5/3/1947	832
1948	3/3/1948	4,469	3/5/1948	2,287	3/8/1948	1,243	3/16/1948	892	3/31/1948	697
1949	2/6/1949	2,683	2/6/1949	2,209	2/10/1949	1,469	2/18/1949	902	2/15/1949	750
1950	11/18/1949	9,390	11/20/1949	6,320	11/23/1949	3,377	12/17/1949	1,913	12/16/1949	1,788
1951	11/18/1950	9,390	11/20/1950	6,320	11/23/1950	3,377	12/17/1950	1,913	12/16/1950	1,788
1952	1/15/1952	7,610	1/16/1952	4,819	1/18/1952	3,484	1/26/1952	2,415	1/26/1952	1,821
1953	1/14/1953	1,992	1/15/1953	1,273	1/19/1953	909	1/21/1953	698	1/28/1953	510
1954	2/14/1954	1,717	2/15/1954	1,097	2/19/1954	809	3/30/1954	693	4/7/1954	558
1955	1/1/1955	2,095	1/20/1955	1,078	1/22/1955	701	1/30/1955	435	1/30/1955	373

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1956	12/23/1955	20,156	12/24/1955	13,299	12/28/1955	7,493	1/5/1956	4,134	1/17/1956	2,864
1957	3/6/1957	7,446	3/7/1957	5,410	3/8/1957	3,072	3/10/1957	2,031	3/24/1957	1,185
1958	4/3/1958	32,920	4/4/1958	22,402	4/7/1958	16,071	4/11/1958	9,898	4/13/1958	6,617
1959	2/11/1959	5,823	2/20/1959	3,446	2/22/1959	2,779	2/25/1959	2,128	3/11/1959	1,314
1960	2/8/1960	4,099	2/10/1960	2,779	2/14/1960	1,426	2/15/1960	789	2/23/1960	452
1961	3/17/1961	277	3/18/1961	232	3/21/1961	175	3/29/1961	142	4/13/1961	96
1962	2/15/1962	7,377	2/16/1962	4,116	2/16/1962	3,053	2/22/1962	1,894	3/10/1962	1,323
1963	2/1/1963	9,416	2/2/1963	6,079	2/5/1963	2,854	2/14/1963	1,547	4/26/1963	1,205
1964	1/22/1964	2,623	1/23/1964	1,828	1/27/1964	1,041	2/3/1964	612	2/17/1964	359
1965	12/23/1964	12,789	12/24/1964	8,666	12/28/1964	5,504	1/6/1965	3,902	1/17/1965	2,722
1966	12/30/1965	2,020	12/31/1965	1,720	1/3/1966	984	1/8/1966	626	1/23/1966	369
1967	1/22/1967	6,738	1/23/1967	3,991	4/24/1967	2,900	2/4/1967	2,172	4/29/1967	1,832
1968	2/21/1968	1,647	2/22/1968	1,301	2/23/1968	938	3/1/1968	560	3/17/1968	435
1969	1/21/1969	14,674	1/22/1969	9,511	1/26/1969	7,000	2/2/1969	4,579	2/17/1969	3,103
1970	1/21/1970	7,200	1/16/1970	5,072	1/22/1970	3,548	1/28/1970	2,852	2/8/1970	1,642
1971	12/2/1970	2,983	12/4/1970	2,256	12/5/1970	1,967	12/12/1970	1,176	12/27/1970	929
1972	12/25/1971	4,922	12/27/1971	2,366	12/28/1971	1,486	1/4/1972	791	1/18/1972	434
1973	1/16/1973	7,695	2/12/1973	5,936	2/16/1973	3,730	2/18/1973	2,268	2/14/1973	1,842
1974	3/2/1974	9,124	3/3/1974	4,946	3/7/1974	2,738	3/15/1974	1,722	3/30/1974	1,101
1975	3/25/1975	5,783	3/27/1975	3,401	3/27/1975	2,538	3/28/1975	1,732	4/5/1975	1,259
1976	3/2/1976	240	3/3/1976	176	3/6/1976	128	3/13/1976	91	3/13/1976	74
1977	3/16/1977	112	11/14/1976	63	2/27/1977	38	3/21/1977	29	3/22/1977	28
1978	3/5/1978	5,770	3/6/1978	4,322	1/20/1978	2,622	1/19/1978	1,734	3/7/1978	1,329
1979	2/22/1979	5,388	2/23/1979	4,643	2/25/1979	2,827	3/4/1979	2,183	3/15/1979	1,441
1980	1/14/1980	8,648	1/14/1980	7,385	1/18/1980	4,744	1/24/1980	2,630	3/15/1980	1,630

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1981	1/29/1981	3,160	1/30/1981	2,148	2/2/1981	1,229	2/5/1981	654	4/2/1981	414
1982	1/5/1982	12,321	2/17/1982	9,059	4/4/1982	4,845	4/12/1982	3,808	4/14/1982	2,648
1983	3/13/1983	10,433	3/2/1983	7,318	3/5/1983	4,913	3/14/1983	3,738	3/27/1983	3,108
1984	12/25/1983	8,029	12/27/1983	5,712	12/30/1983	3,712	1/6/1984	2,099	1/1/1984	1,407
1985	2/8/1985	3,769	2/10/1985	1,892	2/14/1985	953	2/22/1985	511	4/4/1985	416
1986	2/17/1986	23,494	2/19/1986	17,022	2/21/1986	11,280	2/27/1986	5,752	3/16/1986	3,858
1987	3/6/1987	1,761	3/7/1987	1,201	3/11/1987	619	3/19/1987	455	4/3/1987	303
1988	1/17/1988	403	1/18/1988	285	1/21/1988	175	1/24/1988	111	2/3/1988	79
1989	3/25/1989	927	3/27/1989	725	3/30/1989	465	3/16/1989	324	3/31/1989	319
1990	2/17/1990	695	2/18/1990	558	2/22/1990	352	3/17/1990	277	3/17/1990	271
1991	3/26/1991	3,939	3/26/1991	2,955	3/28/1991	1,721	4/1/1991	1,091	4/2/1991	666
1992	2/15/1992	5,114	2/15/1992	2,611	2/17/1992	1,938	2/25/1992	1,180	3/11/1992	747
1993	1/13/1993	5,317	1/15/1993	3,831	1/19/1993	3,063	1/21/1993	2,398	1/27/1993	1,538
1994	2/20/1994	909	2/20/1994	722	2/24/1994	531	3/3/1994	340	3/7/1994	242
1995	3/11/1995	10,146	3/12/1995	8,592	3/15/1995	4,792	3/24/1995	3,896	4/1/1995	2,406
1996	2/21/1996	5,653	2/22/1996	4,658	2/25/1996	3,009	3/5/1996	1,991	2/23/1996	1,527
1997	1/2/1997	16,801	1/3/1997	10,759	1/5/1997	6,316	1/4/1997	4,465	1/28/1997	3,273
1998	2/3/1998	16,919	2/4/1998	8,069	2/8/1998	6,548	2/16/1998	4,317	2/27/1998	3,000
1999	2/9/1999	9,084	2/9/1999	5,840	2/13/1999	3,457	2/21/1999	2,361	3/8/1999	1,560
2000	2/14/2000	7,667	2/14/2000	5,974	2/17/2000	3,534	2/25/2000	2,503	3/11/2000	1,965
2001	3/5/2001	2,094	3/6/2001	1,303	3/9/2001	771	3/6/2001	623	3/11/2001	497
2002	1/3/2002	2,027	1/4/2002	1,439	1/4/2002	1,241	1/4/2002	710	1/12/2002	452
2003	12/16/2002	1,488	12/18/2002	1,087	12/21/2002	685	12/30/2002	438	5/11/2003	339
2004	2/26/2004	3,011	2/28/2004	2,039	3/2/2004	1,246	3/3/2004	779	3/16/2004	484
2005	3/23/2005	10,277	3/24/2005	6,101	3/28/2005	3,614	1/13/2005	2,286	1/28/2005	1,384

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
2006	4/4/2006	18,294	4/5/2006	12,106	4/7/2006	7,121	4/8/2006	4,518	4/23/2006	3,101
2007	2/27/2007	2,715	2/28/2007	1,937	3/3/2007	1,147	3/8/2007	652	3/10/2007	468
2008	1/28/2008	2,313	1/29/2008	1,309	2/3/2008	995	2/6/2008	843	2/21/2008	494
2009	3/4/2009	4,310	3/5/2009	2,592	3/8/2009	1,470	3/9/2009	902	3/14/2009	629
2010	1/22/2010	3,054	1/22/2010	2,547	1/25/2010	1,591	2/1/2010	904	2/16/2010	580

Table 18. Calaveras River at Bellota annual maximum series for unregulated volume-frequency analysis

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1907	3/19/1907	28,983	3/19/1907	16,561	3/23/1907	11,383	3/24/1907	8,661	4/2/1907	5,578
1908	2/10/1908	2,486	2/11/1908	1,375	2/14/1908	760	1/28/1908	580	2/12/1908	526
1909	1/21/1909	21,914	1/22/1909	10,038	1/26/1909	6,345	1/27/1909	5,485	2/12/1909	4,137
1910	12/9/1909	8,766	12/9/1909	4,100	12/11/1909	2,572	12/15/1909	1,793	1/3/1910	1,126
1911	1/31/1911	36,995	2/1/1911	25,119	1/31/1911	13,101	2/3/1911	8,231	2/10/1911	5,397
1912	3/13/1912	1,320	3/15/1912	787	3/19/1912	589	3/20/1912	453	4/4/1912	305
1913	1/19/1913	1,567	1/19/1913	955	1/21/1913	683	1/29/1913	422	2/13/1913	248
1914	2/21/1914	10,722	2/21/1914	7,576	1/28/1914	4,869	1/28/1914	3,424	1/29/1914	2,362
1915	2/1/1915	9,920	2/3/1915	8,487	2/3/1915	5,492	2/11/1915	4,425	2/26/1915	2,844
1916	3/20/1916	11,699	3/22/1916	5,541	1/30/1916	3,651	1/28/1916	3,180	2/7/1916	2,694
1917	2/21/1917	23,210	2/23/1917	16,848	2/27/1917	10,178	3/6/1917	5,678	3/20/1917	3,346
1918	3/11/1918	19,911	3/12/1918	14,390	3/13/1918	8,141	3/21/1918	4,732	3/24/1918	2,795
1919	2/11/1919	8,766	2/12/1919	4,662	2/16/1919	2,260	2/24/1919	1,252	3/11/1919	1,041
1920	3/17/1920	3,499	3/23/1920	2,926	3/22/1920	2,340	3/24/1920	1,549	3/30/1920	1,023
1921	1/18/1921	28,983	1/20/1921	13,416	1/23/1921	6,438	1/31/1921	4,006	2/15/1921	2,392
1922	2/20/1922	11,063	2/11/1922	9,327	2/14/1922	4,748	2/23/1922	3,762	3/9/1922	2,211
1923	12/13/1922	8,283	12/14/1922	6,417	12/16/1922	3,594	12/21/1922	2,001	1/5/1923	1,340
1924	2/6/1924	212	2/8/1924	198	2/12/1924	129	2/15/1924	99	2/15/1924	74
1925	2/6/1925	15,552	2/7/1925	7,764	2/10/1925	4,041	2/18/1925	2,541	3/5/1925	1,679
1926	2/14/1926	3,605	2/14/1926	3,075	2/18/1926	1,831	2/17/1926	1,199	2/28/1926	788
1927	2/4/1927	7,046	2/5/1927	4,285	2/21/1927	3,153	2/18/1927	2,033	3/4/1927	1,662
1928	3/25/1928	12,607	3/26/1928	7,957	3/30/1928	5,133	4/7/1928	2,907	4/22/1928	1,611
1929	2/4/1929	1,909	2/5/1929	1,201	2/8/1929	709	2/15/1929	399	2/17/1929	267
1930	3/6/1930	3,719	3/7/1930	3,364	3/10/1930	1,966	3/9/1930	1,320	3/23/1930	814

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1931	2/15/1931	927	2/16/1931	522	2/21/1931	418	2/28/1931	267	3/15/1931	183
1932	12/28/1931	12,285	2/9/1932	9,182	2/11/1932	6,107	2/14/1932	4,291	2/28/1932	2,386
1933	1/28/1933	1,959	1/30/1933	1,807	1/30/1933	1,373	2/3/1933	807	2/18/1933	542
1934	12/30/1933	6,058	1/1/1934	4,090	1/4/1934	2,838	3/5/1934	1,518	3/9/1934	927
1935	3/8/1935	7,430	4/10/1935	6,052	4/10/1935	4,358	4/17/1935	3,121	5/2/1935	1,997
1936	2/23/1936	28,648	2/24/1936	23,679	2/26/1936	13,565	2/26/1936	12,451	3/2/1936	7,023
1937	2/6/1937	19,366	2/7/1937	16,090	2/10/1937	8,591	2/16/1937	5,853	2/27/1937	3,766
1938	2/11/1938	33,263	2/12/1938	22,349	2/16/1938	14,296	2/15/1938	9,795	3/4/1938	6,030
1939	2/8/1939	2,522	2/9/1939	1,406	2/13/1939	816	2/14/1939	546	2/28/1939	372
1940	3/4/1940	13,646	2/29/1940	11,312	3/4/1940	8,606	3/8/1940	5,011	3/4/1940	2,966
1941	4/4/1941	10,534	3/3/1941	7,072	3/6/1941	4,994	3/8/1941	3,128	3/9/1941	2,765
1942	1/27/1942	17,509	1/28/1942	12,913	1/30/1942	8,951	2/7/1942	5,797	2/21/1942	3,398
1943	1/21/1943	13,940	1/23/1943	10,340	3/11/1943	8,966	3/19/1943	6,061	3/25/1943	4,122
1944	2/3/1944	6,587	2/5/1944	4,528	2/8/1944	2,707	2/16/1944	1,684	3/2/1944	1,090
1945	12/23/1944	4,259	12/24/1944	3,373	12/28/1944	2,781	1/5/1945	1,905	1/19/1945	1,200
1946	12/21/1945	1,338	3/12/1946	983	3/16/1946	658	3/18/1946	451	4/8/1946	423
1947	3/25/1947	1,562	4/8/1947	1,075	4/25/1947	947	5/2/1947	890	5/3/1947	833
1948	3/3/1948	4,469	3/5/1948	2,287	3/8/1948	1,244	3/17/1948	900	3/31/1948	749
1949	2/6/1949	2,704	2/6/1949	2,236	2/10/1949	1,495	2/18/1949	919	2/15/1949	762
1950	11/18/1949	9,390	11/20/1949	6,320	11/23/1949	3,377	12/17/1949	1,913	12/16/1949	1,788
1951	11/18/1950	11,646	11/20/1950	7,694	12/9/1950	4,212	12/17/1950	2,490	12/17/1950	2,245
1952	1/15/1952	8,449	1/16/1952	5,405	1/18/1952	3,985	1/26/1952	2,855	1/26/1952	2,139
1953	1/14/1953	2,191	1/15/1953	1,402	1/19/1953	1,067	1/21/1953	832	1/28/1953	603
1954	2/14/1954	1,986	2/15/1954	1,228	2/19/1954	903	3/30/1954	751	4/7/1954	601
1955	1/1/1955	2,735	1/20/1955	1,681	1/21/1955	1,101	1/29/1955	645	1/30/1955	527

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1956	12/23/1955	22,716	12/24/1955	14,792	12/28/1955	8,324	1/5/1956	4,610	1/17/1956	3,254
1957	3/6/1957	7,737	3/7/1957	6,001	3/8/1957	3,413	3/10/1957	2,235	3/24/1957	1,298
1958	4/3/1958	34,868	4/4/1958	24,018	4/7/1958	17,188	4/11/1958	10,513	4/13/1958	7,085
1959	2/11/1959	6,252	2/19/1959	3,826	2/22/1959	3,109	2/25/1959	2,342	3/11/1959	1,434
1960	2/8/1960	4,233	2/10/1960	2,898	2/14/1960	1,485	2/15/1960	816	2/23/1960	466
1961	3/17/1961	299	3/18/1961	246	3/21/1961	183	3/29/1961	148	4/13/1961	99
1962	2/15/1962	8,141	2/16/1962	4,601	2/16/1962	3,493	2/23/1962	2,140	3/10/1962	1,505
1963	2/1/1963	10,568	2/2/1963	6,670	2/5/1963	3,128	2/14/1963	1,735	4/26/1963	1,341
1964	1/22/1964	3,045	1/23/1964	2,233	1/27/1964	1,242	2/2/1964	715	2/16/1964	414
1965	12/23/1964	14,895	12/24/1964	9,950	12/28/1964	6,263	1/6/1965	4,333	1/17/1965	3,012
1966	12/30/1965	2,276	12/31/1965	1,940	1/3/1966	1,110	1/8/1966	700	2/27/1966	412
1967	1/22/1967	7,813	1/23/1967	4,760	4/24/1967	3,303	2/4/1967	2,635	4/29/1967	2,092
1968	2/21/1968	2,133	2/22/1968	1,626	2/23/1968	1,113	3/1/1968	651	3/17/1968	503
1969	1/21/1969	15,548	1/21/1969	10,261	1/26/1969	7,612	2/2/1969	4,996	2/17/1969	3,446
1970	1/21/1970	8,827	1/16/1970	6,218	1/22/1970	4,350	1/28/1970	3,496	2/8/1970	2,014
1971	12/2/1970	3,657	12/4/1970	2,765	12/5/1970	2,412	12/12/1970	1,441	12/27/1970	1,139
1972	12/25/1971	6,034	12/27/1971	2,901	12/28/1971	1,822	1/4/1972	969	1/18/1972	532
1973	1/16/1973	9,434	2/12/1973	7,278	2/16/1973	4,573	2/18/1973	2,781	2/14/1973	2,259
1974	3/2/1974	11,186	3/3/1974	6,064	3/7/1974	3,357	3/15/1974	2,111	3/30/1974	1,350
1975	3/25/1975	7,090	3/27/1975	4,169	3/27/1975	3,112	3/28/1975	2,124	4/5/1975	1,543
1976	3/2/1976	294	3/3/1976	216	3/6/1976	157	3/13/1976	111	3/13/1976	90
1977	3/16/1977	137	11/14/1976	77	2/27/1977	47	3/21/1977	36	3/22/1977	34
1978	3/5/1978	7,074	3/6/1978	5,299	1/20/1978	3,214	1/19/1978	2,126	3/7/1978	1,629
1979	2/22/1979	6,606	2/23/1979	5,693	2/25/1979	3,466	3/4/1979	2,676	3/15/1979	1,766
1980	1/14/1980	10,602	1/14/1980	9,054	1/18/1980	5,816	1/24/1980	3,224	3/15/1980	1,999

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
1981	1/29/1981	3,874	1/30/1981	2,633	2/2/1981	1,507	2/5/1981	802	4/2/1981	508
1982	1/5/1982	15,106	2/17/1982	11,106	4/4/1982	5,940	4/12/1982	4,669	4/14/1982	3,247
1983	3/13/1983	12,791	3/2/1983	8,972	3/5/1983	6,024	3/14/1983	4,583	3/27/1983	3,811
1984	12/25/1983	9,844	12/27/1983	7,003	12/30/1983	4,551	1/6/1984	2,573	1/1/1984	1,726
1985	2/8/1985	4,621	2/10/1985	2,320	2/14/1985	1,168	2/22/1985	627	4/4/1985	509
1986	2/17/1986	28,804	2/19/1986	20,869	2/21/1986	13,830	2/27/1986	7,052	3/16/1986	4,730
1987	3/6/1987	2,159	3/7/1987	1,472	3/11/1987	759	3/19/1987	558	4/3/1987	371
1988	4/22/1988	8,595	4/24/1988	8,126	4/26/1988	7,278	4/26/1988	5,733	4/27/1988	5,231
1989	3/25/1989	1,137	3/27/1989	817	3/30/1989	522	3/16/1989	398	3/31/1989	380
1990	3/3/1990	1,167	3/5/1990	709	3/9/1990	561	3/11/1990	425	3/17/1990	360
1991	5/14/1991	7,875	5/15/1991	6,864	5/19/1991	4,914	5/23/1991	3,156	6/11/1991	1,742
1992	2/15/1992	6,982	5/8/1992	3,447	5/9/1992	3,013	6/12/1992	2,669	6/26/1992	1,695
1993	5/5/1993	7,550	5/6/1993	7,021	5/6/1993	5,450	5/6/1993	3,330	1/27/1993	1,857
1994	10/7/1993	1,705	2/20/1994	885	2/24/1994	652	3/3/1994	417	3/7/1994	296
1995	3/11/1995	12,439	3/12/1995	10,533	3/15/1995	5,875	3/24/1995	4,777	4/1/1995	2,950
1996	2/21/1996	6,569	2/22/1996	5,185	2/25/1996	3,251	3/5/1996	2,133	2/23/1996	1,670
1997	1/2/1997	20,116	1/3/1997	13,031	1/5/1997	7,579	1/4/1997	5,455	1/29/1997	3,868
1998	2/3/1998	22,236	2/5/1998	10,599	2/9/1998	8,332	2/16/1998	5,470	2/27/1998	3,856
1999	2/9/1999	11,835	2/9/1999	7,401	2/13/1999	4,228	2/21/1999	2,895	3/8/1999	1,911
2000	2/14/2000	9,281	2/14/2000	7,554	2/17/2000	4,336	2/25/2000	2,998	3/11/2000	2,327
2001	3/5/2001	3,167	3/7/2001	1,823	3/9/2001	1,048	3/7/2001	801	3/11/2001	618
2002	1/3/2002	3,431	1/4/2002	2,174	1/4/2002	1,894	1/4/2002	1,032	1/12/2002	621
2003	12/17/2002	1,920	12/18/2002	1,337	12/21/2002	810	12/30/2002	503	1/13/2003	358
2004	2/27/2004	4,806	2/28/2004	3,147	3/2/2004	1,827	3/3/2004	1,084	3/16/2004	639
2005	3/23/2005	12,358	3/24/2005	7,321	3/28/2005	4,321	1/13/2005	2,950	1/29/2005	1,796

Water year (1)	Date of 1-day max volume (2)	1-day max volume (cfs) (3)	Date of 3-day max volume (4)	3-day max volume (cfs) (5)	Date of 7-day max volume (6)	7-day max volume (cfs) (7)	Date of 15-day max volume (8)	15-day max volume (cfs) (9)	Date of 30-day max volume (10)	30-day max volume (cfs) (11)
2006	4/4/2006	21,665	4/6/2006	14,613	4/7/2006	8,540	4/12/2006	5,409	4/23/2006	3,657
2007	2/27/2007	3,081	2/28/2007	2,147	3/3/2007	1,237	3/8/2007	695	3/10/2007	489
2008	1/28/2008	2,870	1/29/2008	1,553	2/3/2008	1,148	2/6/2008	968	2/21/2008	557
2009	3/4/2009	4,956	3/5/2009	2,935	3/8/2009	1,623	3/9/2009	980	3/14/2009	693
2010	1/20/2010	4,467	1/22/2010	3,664	1/25/2010	2,166	2/1/2010	1,225	2/16/2010	774

Peak annual maximum series

To develop the peak inflow annual maximum series for New Hogan Reservoir, we reviewed the data provided by the Corps and other sources that contain annual maximum series, including:

- New Hogan Reservoir water control manual (USACE 1983a), hereafter referred to as New Hogan WCM.
- Calaveras River reconnaissance report (USACE 1990).
- Peak flow data provided by the Corps on 6/11/2010.

We summarize in Table 19 the data we identified for use in developing flow-frequency curves for New Hogan. Column 1 lists the time period for which data were identified, and column 2 lists the source of these data.

Table 19. Data sources of peak inflow annual maximum series data identified for use in developing flow-frequency curves for New Hogan Reservoir

Time period (water year) (1)	Data source used (2)
1907-1929[1]	Data provided by Corps on 6/11/2010
1930-1979[2]	New Hogan WCM (USACE 1983a)
1980-1988	Calaveras River reconnaissance report (USACE 1990)
1989-2010	Data provided by Corps on 6/11/2010

Notes:

1. Data missing for the 1924 water year.
2. Data missing for the periods 1944-1955, 1960-1963, and 1970 water years.

We list the peak inflow values and, where possible, their associated dates of occurrence, for New Hogan Reservoir in Table 20. In the table, column 1 lists the water year; column 2 lists the date, if available; and column 3 lists the value in cfs.

We did not develop a peak flow-frequency curve for the Calaveras River at Bellota because a series of annual maximum peak flows at this location is not available. A peak unregulated flow-frequency curve is not required for this analysis.

Table 20. New Hogan Reservoir annual maximum peak inflows

Water year (1)	Date of peak inflow (2)	Peak inflow (cfs) (3)
1907	3/19/1907	34,600
1908	2/10/1908	2,110
1909	1/21/1909	33,000
1910	12/9/1910	11,200
1911	1/31/1911	50,000
1912	3/13/1912	1,120
1913	1/19/1913	1,330
1914	1/22/1914	12,100
1915	2/2/1915	9,190
1916	1/17/1916	22,000
1917	2/21/1917	31,300
1918	3/12/1918	21,800
1919	2/11/1919	11,000
1920	3/17/1920	2,970
1921	1/18/1921	37,900
1922	2/9/1922	24,500
1923	12/13/1923	7,030
1924	—	—
1925	2/6/1925	27,500
1926	2/13/1926	12,700
1927	2/3/1927	19,300
1928	3/25/1928	17,300
1929	2/4/1929	3,060
1930	3/5/1930	10,500
1931	2/15/1931	860
1932	2/6/1932	13,000
1933	1/29/1933	2,060
1934	1/1/1934	4,800
1935	3/7/1935	11,000
1936	2/22/1936	35,000
1937	2/6/1937	14,000
1938	2/11/1938	41,000
1939	2/7/1939	1,780
1940	2/27/1940	18,000
1941	4/4/1941	10,800
1942	1/27/1942	18,300
1943	3/6/1943	14,900
1944-1955	—	—

Water year (1)	Date of peak inflow (2)	Peak inflow (cfs) (3)
1956	12/23/1955	31,500
1957	3/6/1957	7,912
1958	4/2/1958	42,000
1959	2/11/1959	6,640
1960-1963	—	—
1964	1/22/1964	4,820
1965	12/23/1964	20,600
1966	12/30/1965	3,720
1967	1/21/1967	17,500
1968	2/21/1968	3,040
1969	1/21/1969	19,300
1970	—	—
1971	12/2/1970	5,480
1972	12/25/1971	9,050
1973	1/16/1973	13,500
1974	3/2/1974	18,000
1975	3/25/1975	9,650
1976	3/2/1976	440
1977	3/16/1977	200
1978	3/5/1978	10,600
1979	2/22/1979	9,940
1980	1/14/1980	17,900
1981	1/29/1981	6,500
1982	3/31/1982	23,600
1983	3/13/1983	19,454
1984	12/25/1983	10,440
1985	2/8/1985	7,100
1986	2/19/1986	32,444
1987	3/6/1987	3,055
1988	1/17/1988	800
1989	3/25/1989	1,467
1990	2/17/1990	1,135
1991	3/26/1991	10,003
1992	2/15/1992	10,581
1993	1/18/1993	11,572
1994	1/22/1994	2,108
1995	3/10/1995	19,616
1996	2/21/1996	9,070
1997	1/3/1997	23,920

Water year (1)	Date of peak inflow (2)	Peak inflow (cfs) (3)
1998	2/3/1998	33,055
1999	2/9/1999	16,129
2000	1/25/2000	13,762
2001	3/5/2001	3,375
2002	1/2/2002	4,221
2003	12/16/2002	4,010
2004	1/1/2004	5,423
2005	3/23/2005	12,107
2006	4/4/2006	25,555
2007	2/26/2007	5,688
2008	1/28/2008	4,490
2009	3/4/2009	9,424
2010	12/25/2009	13,785

Attachment 4: Fitting the unregulated frequency curves

Overview

The purpose of this attachment is to describe the steps taken to fit unregulated frequency curves to annual maximum series. We developed unregulated frequency curves following the procedures specified in *Bulletin 17B* (IACWD 1982), guidance detailed in EM 1110-2-1415 (USACE 1993), and the current standards of practice. Specifically, we:

- Identified the annual maximum series.
- (Task 4.1) Calculated regional skew values for each duration of interest using relationships developed by the USGS.
- (Task 4.2) Fitted LPIII distributions to the annual maximum series following *Bulletin 17B* procedures and Corps guidance using PeakfqSA, the USGS's flow-frequency software with the expected moments algorithm (EMA) option enabled developed by Tim Cohn of the USGS (Cohn 2007).
- Reviewed and adopted the curves, checking them for consistency and comparing them to previously accepted values.

Regional skew values

Bulletin 17B recommends the use of a regional skew value in fitting LPIII distributions to maintain consistency of frequency curves. *Bulletin 17B* also states that such a value can be developed using regression techniques. For the CVHS, the USGS, in cooperation with the Corps, has developed regression equations for regional skew values (USGS 2010). In general, there are 2 equation forms, 1 for peak flows, and 1 for volumes. The coefficients for the volumes change with duration.

The regional skew associated with peak flows is calculated as:

$$\gamma = -0.62 + 1.30\left(1 - e^{\left(-\left(\frac{Elev}{6500}\right)^{2}\right)}\right) \qquad (5)$$

where γ is the regional skew value and *Elev* is the average basin elevation in ft (NAVD 88). The associated average variance of prediction (AVP) is 0.14. AVP is analogous to mean square error (MSE) for the purpose of weighting regional and station skew values.

The regional skew associated with volumes is calculated as

$$\gamma = \beta_0 + \beta_1\left(1 - e^{\left(-\left(\frac{Elev}{3600}\right)^{12}\right)}\right) \qquad (6)$$

where γ is the regional skew value, *Elev* is the average basin elevation in ft (NAVD 88), and β_0 and β_1 are coefficients based on the duration of interest as shown in Table 21. The associated AVP also varies with duration and is also shown in Table 21.

For this analysis, we used these equations to develop regional skew values for the Calaveras River as shown in Table 22. We used GIS tools to compute average basin elevations for use in the regional skew computations.

Table 21. Duration skew equation parameters

Parameter (1)	1-day regional skew (2)	3-day regional skew (3)	7-day regional skew (4)	15-day regional skew (5)	30-day regional skew (6)
β_0	-0.7340	-0.6901	-0.5872	-0.6445	-0.6322
β_1	0.6778	0.6764	0.5822	0.5375	0.4277
AVP	0.0485	0.0576	0.0490	0.0521	0.0615

Table 22. Regional skew values

Location (1)	Elevation (ft) (2)	Peak flow regional skew (3)	1-day regional skew (4)	3-day regional skew (5)	7-day regional skew (6)	15-day regional skew (7)	30-day regional skew (8)
New Hogan Reservoir	2010.31	-0.501	-0.733	-0.690	-0.587	-0.644	-0.632
Bellota	1662.53	-0.538	-0.734	-0.690	-0.587	-0.644	-0.632

Fitting the curves

As a first step, the curves were fitted using a straightforward *Bulletin 17B* procedure in which all data points were included in the analysis and low outliers were identified by the *Bulletin 17B* outlier test and the station statistics appropriately adjusted. This includes weighting the station skew and regional skew values by the inverse of their associated errors. This weighting procedure is included in *Bulletin 17B* and the weighted skew is automatically calculated by PeakfqSA, which we used here.

We found the frequency curves on the Calaveras River were consistent between durations at each location. The curves do not "cross," and flow quantiles for a given duration at the downstream location were greater than those of the upstream location, as would be expected.

As a comparison, we considered the volume-frequency curves developed for Farmington Reservoir in the Comp Study (USACE 2002). The annual maximum series in the Comp Study ended in 1997.

We then compared the curves fitted at New Hogan Reservoir to the corresponding curves from the Comp Study (USACE 2002). We found that the flow quantiles of the curves fitted here and those of the Comp Study differ between the 2 sets of volume-duration curves by only 1%-13%. The greatest differences (of only 8%-13%) are in the 1-day volume quantiles. The 3-day and 7-day volume quantiles differ by only 1% to 5%. Peak flow-frequency curves varied by as much as 9% because of the increased number of large events included in this analysis as compared to the Comp Study.

Results

The final parameters and statistics used to fit LPIII distributions to develop the unregulated frequency curves at New Hogan Reservoir (shown in Figure 9) are shown in Table 23.

The final parameters and statistics used to fit LPIII distributions to develop the unregulated frequency curves at Bellota (shown in Figure 10) are shown in Table 24.

Table 23. Parameters and statistics to fit unregulated frequency curves: New Hogan Reservoir

Statistic (1)	Peak flows (2)	1-day volumes (3)	3-day volumes (4)	7-day volumes (5)	15-day volumes (6)	30-day volumes (7)
Station mean[1]	3.946	3.684	3.518	3.324	3.146	2.988
Station standard deviation[1]	0.485	0.502	0.488	0.478	0.473	0.459
Station skew[1]	-1.027	-0.979	-0.819	-0.806	-0.682	-0.706
Station skew associated MSE[2]	0.160	0.126	0.107	0.105	0.093	0.095
Regional skew[3]	-0.501	-0.733	-0.690	-0.587	-0.644	-0.632
Regional skew associated AVP[4]	0.140	0.049	0.058	0.049	0.052	0.062
Mean[5]	3.947	3.685	3.518	3.324	3.146	2.988
Standard deviation[5]	0.482	0.501	0.488	0.477	0.473	0.458
Weighted skew[5,6]	-0.727	-0.794	-0.731	-0.651	-0.646	-0.659
Number of systematic events	86	104	104	104	104	104
Number of high outliers	0	0	0	0	0	0
Number of EMA iterations	2	2	2	2	2	2
Number of low outliers	0	2	2	2	2	2
Number of zero events	0	0	0	0	0	0
Number of missing events	18	0	0	0	0	0
Number of EMA censored observations	1	1	1	1	1	1
Corresponding censored events[7]	1). 1977	1). 1977	1). 1977	1). 1977	1). 1977	1). 1977
Record length	104	104	104	104	104	104

Notes:

1. Statistic calculated using the series of logarithmic transforms and EMA without regional skew; rounded to nearest thousandth.
2. Mean square error; rounded to nearest thousandth.
3. Regional skew values calculated using relationships developed by the USGS; rounded to nearest thousandth.
4. Average variance of prediction, analogous to MSE; rounded to nearest thousandth.
5. Statistic calculated using the series of logarithmic transforms and EMA with regional skew; rounded to nearest thousandth.
6. Skew value calculated by weighting the station and regional skew values inversely proportional to their associated errors: (MSE and AVP) and EMA; rounded to nearest thousandth.
7. Events are listed by water year in order of increasing flow or volume.

Table 24. Parameters and statistics to fit unregulated frequency curves: Bellota

Statistic (1)	1-day volumes (2)	3-day volumes (3)	7-day volumes (4)	15-day volumes (5)	30-day volumes (6)
Station mean[1]	3.774	3.607	3.417	3.239	3.079
Station standard deviation[1]	0.487	0.476	0.465	0.461	0.448
Station skew[1]	-1.112	-0.898	-0.875	-0.729	-0.731
Station skew associated MSE[2]	0.145	0.116	0.113	0.097	0.096
Regional skew[3]	-0.734	-0.690	-0.587	-0.644	-0.632
Regional skew associated AVP[4]	0.049	0.058	0.049	0.052	0.062
Mean[5]	3.775	3.608	3.417	3.240	3.079
Standard deviation[5]	0.482	0.475	0.464	0.461	0.448
Weighted skew[5,6]	-0.810	-0.753	-0.666	-0.671	-0.668
Number of systematic events	104	104	104	104	104
Number of high outliers	0	0	0	0	0
Number of EMA iterations	2	2	2	2	2
Number of low outliers	0	0	0	0	0
Number of zero events	0	0	0	0	0
Number of missing events	0	0	0	0	0
Number of EMA censored observations	2	1	1	1	1
Corresponding censored events[7]	1). 1977 2). 1976	1). 1977	1). 1977	1). 1977	1). 1977
Record length	104	104	104	104	104

Notes:

1. Statistic calculated using the series of logarithmic transforms and EMA without regional skew; rounded to nearest thousandth.
2. Mean square error; rounded to nearest thousandth.
3. Regional skew values calculated using relationships developed by the USGS; rounded to nearest thousandth.
4. Average variance of prediction, analogous to MSE; rounded to nearest thousandth.
5. Statistic calculated using the series of logarithmic transforms and EMA with regional skew; rounded to nearest thousandth.
6. Skew value calculated by weighting the station and regional skew values inversely proportional to their associated errors: (MSE and AVP) and EMA; rounded to nearest thousandth.
7. Events are listed by water year in order of increasing volume.

Attachment 5: Unregulated-regulated flow transforms and critical duration assessment

Fit unregulated-regulated flow transforms

We developed the unregulated-regulated flow transforms for the 2 analysis locations by fitting transform curves through data pairs from the event maxima datasets. Specifically, we fitted transforms to pairs of unregulated volumes (as average flows) and regulated peak flows. For this analysis, we used unregulated volumes associated with the 1-, 1.5-, 2-, 2.5-, 3-, 3.5-, 4-, 4.5-, 5-, 6-, 7-, 10-, 15-, and 30-day durations. We fitted these curves to the data pairs of historical and scaled events using the robust locally weighted scatterplot smoothing (LOWESS) regression technique. (The LOWESS procedure is detailed in the *Technical procedure document*.)

Here, we used the LOWESS algorithm developed by William Cleveland (Cleveland 1985). We complied an executable of the algorithm, implemented in Fortran. This executable was tested using example data included in the Fortran file.

We used an iterative process to fit these transforms. Specifically we:

- Fitted a candidate transform using the LOWESS regression technique.
- Calculated the mean squared error (MSE) associated with the candidate transform.
- Modified the LOWESS parameters using guidance provided in the literature (Bradley and Potter 2004, Cleveland 1979).
- Fitted another candidate transform and calculated the associated MSE.
- Compared this new transform to the old transform(s) visually and based on MSE.
- Repeated the previous steps until the parameters resulting in the best fit, as determined visually and based on MSE, were identified.

Determine critical duration

For a regulated system, the critical duration is the unregulated flow duration-frequency curve that best characterizes the peak regulated flow-frequency curve at a downstream point. To determine critical duration for each location, we:

- Fitted flow transforms to the event maxima datasets, as detailed in the previous subsection.
- Applied these flow transforms to develop hypothetical regulated flow-frequency curves.
- Identified the duration of the unregulated annual maximum series that estimates the largest flow for each probability of interest, as shown in column 1 of Table 25. Here, we considered 2 criteria: (1) the "goodness of fit" of each transform, and (2) which duration estimates the greater peak regulated flows

Table 25. Synthesis of information used to determine critical duration

Annual exceedence probability (1)	Unregulated flow duration (in days) that estimates the largest flow quantile at New Hogan Reservoir (2)	Unregulated flow duration (in days) that estimates the largest flow quantile at Bellota (3)
0.500	2.5	3.5
0.200	1	3
0.100	1	3
0.050	1	2
0.020	1	1
0.010	5	1
0.005	3.5	1
0.002	3.5	2.5

After considering all the durations noted above, for New Hogan Reservoir, we focused on durations of 10 days or less because: (1) the typical unregulated inflow event duration is less than 15 days, and (2) the flow transforms for durations of 10 days or less better fit the event maxima data pairs based on MSE and visual inspection. In addition, the scaled historical event unregulated volumes associated with the longer durations tend to include volumes of additional flood waves after the peak reservoir release. These later flood waves do not contribute to the inflow volumes that drive the reservoir releases, unlike multiple flood waves prior to the peak reservoir releases that are considered. Here, we defined a flood event as the time from when the pool elevation rises from and returns to the top of conversation pool (bottom of flood control pool). For Bellota, we looked at durations equal or less than the critical duration at New Hogan because the addition of unregulated local flows will not cause the critical duration to increase.

In selection of the critical duration, we gave more weight to the durations that estimated the largest flow quantiles for the p=0.01, p=0.005, and p=0.002 annual exceedence events. We used these probabilities because New Hogan Reservoir has large flood storage volume, and regulated peak flows associated with more common events are driven by local flow peaks, not reservoir inflow volumes for a given duration.

From this analysis we determined that the critical duration at New Hogan Reservoir is 3.5 days and at Bellota is 1 day. Thus, the appropriate unregulated-regulated flow transforms used in this analysis were associated with these durations. The critical duration associated with the downstream operation point is shorter than that of the reservoir because of the effects of local flow.

As a "reality check" on our critical duration values, we simulated events, with the HEC-ResSim model, that corresponded to specific volumes associated with a given duration and annual exceedence probability. This is an alternative option for assessing critical duration as detailed in Attachment F of the *Technical Procedures document* as "Method 2: Limited sample, specific volume-duration event scaling." For this check, we scaled reservoir inflows for 4 event patterns (1958, 1986, 1997, and 1998) to the 1-, 3-, 5, and 7-day unregulated flows for the p=0.01 and p=0.005 annual exceedence probabilities. We found: (1) the resulting regulated peaks sensitive to hydrograph shape, and (2) the scaling to the 1-day and 3-day durations estimated largest regulated peak flows. These results are consistent with the adopted critical duration values for the 2 analysis locations.

Review and adopt transforms

After determining the critical duration associated with each analysis location, we reviewed the unregulated-regulated flow transforms initially fitted with the LOWESS procedure to: (1) check for appropriateness, and (2) identify the need for adjustments, if any. As part of this review we:

- Compared event hydrographs of the simulated events that correspond to the transitional areas of the transform (i.e., where the objective peak flows are being constrained, or where peak releases become larger than the objective).
- Fitted additional transforms omitting scaled historical events with scale factors of 2 or less.
- Identified and compared the unregulated volumes that define the "break points" where large floods-of-record and their scaled versions were not controlled by the reservoir because of (1) lack of storage capacity, or (2) local flows larger than the channel capacity.
- Split the unregulated-regulated flow transform initially fitted with LOWESS into 2 ranges using this break point.
- Calculated the MSE for these 2 ranges for each initially fitted LOWESS curve.
- Identified which LOWESS curves have the least MSE for each range.

At New Hogan Reservoir, we found: (1) the LOWESS fitted curves with smoothing coefficients of 0.2 have the lowest MSE for the range of unregulated flows for which the downstream objective flow is met, and (2) the LOWESS fitted curves with smoothing coefficients of 0.5 or greater have a lower MSE for the range in which the downstream objective flow is being exceeded.

Therefore, we blended the 2 "best-fit" LOWESS fitted curves at this break point. We linearly interpolated through the 2 points tangent each curve with the controlling point of tangency nearest to the average "break point" previously identified. We then adjusted the transforms so that the regulated peak flow does not decrease as unregulated volume increases. This blending is seen in Figure 21. In Figure 21 we show the unregulated-regulated flow transforms in black dashes, the floods-of-record event maxima in red squares, the historical scaled event maxima in green triangles, and the initial LOWESS fitted flow transforms in blue and orange for comparison. The blue and orange lines represent the LOWESS fitted curves that best fits event maxima for the more common events and for the more rare events. In Figure 21, the dashed black line represents the recommended transform, including the portion that was blended.

At Bellota, we found that the LOWESS fitted curves with a smoothing coefficient of 0.2 had lowest MSE for ranges of unregulated 1-day volumes both larger and smaller than that associated with the "break point." However, we found that the transform associated with smoothing coefficient of 0.2 does not visually fit the data above this range of interest. Therefore, we completed a sensitivity analysis and found that a smoothing coefficient of 0.24 most appropriately represents this upper range. We blended the 2 "best-fit" LOWESS fitted curves at this point of transition. This blending is seen in Figure 22, in which blue and orange lines represent the LOWESS fitted curves that best fits event maxima for the more common events and for the more rare events.

As a final check, we re-applied the transform to compute the associated regulated flow quantiles. We compared these quantiles to those associated with the original fit, and those associated with the candidate transforms for the other unregulated volumes. For New Hogan Reservoir, we computed a 25% decrease in the p=0.002 quantile. For Bellota, we computed a 1% decrease in the p=0.05 and p=0.02 quantiles. In addition, we re-analyzed the critical duration using the adjusted transform for each analysis location and found them to be consistent with the initial fittings.

Based on this review, we adopted flow transforms for New Hogan Reservoir and Bellota shown in Figure 21 and Figure 22. The tabulated curves are in an MS Excel file on DVD with the original report.

In Figure 21 and Figure 22 we show the unregulated-regulated flow transforms in black, the floods-of-record event maxima in red squares, the historical scaled event maxima in green triangles, and the initial LOWESS fitted flow transforms in blue and orange for comparison. We also show in grey in Figure 21 and Figure 22 the corresponding unregulated volume-duration quantiles for annual exceedence probabilities of interest. In Figure 21 and Figure 22, some scaled historical event maxima for more common events, i.e., annual exceedence probabilities greater than p=0.01, have regulated peaks exceeding the channel capacity (12,500 cfs) because of large local flows.

We show in Table 26 and Table 27 the parameters we used to fit these transforms and the resulting mean square errors. Highlighted in grey in Table 26 and Table 27 are the LOWESS fitted curves with smoothing coefficients listed in column 1 used in fitting the final unregulated-regulated flow transforms over the ranges specified in columns 4 and 5.

Table 26. LOWESS parameters and resulting errors for fitting of unregulated-regulated flow transforms: New Hogan Reservoir

Smoothing coefficient[1] (1)	Number of iterations[2] (2)	Delta[3] (3)	Minimum threshold (1,000 cfs) (4)	Maximum threshold (1,000 cfs) (5)	Total number of data pairs (6)	MSE[4] (7)
0.2	2	0	3	30	250	2,697,208
			3	26	190	312,921
			26	30	60	35,055,387
0.5			3	30	250	2,057,807
			3	26	190	736,368
			26	30	60	19,991,618
Adopted transform			**3**	**30**	**—**	**1,554,705**

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to "save intermediate computations," and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 4 and 5.

Table 27. LOWESS parameters and resulting errors for initial fitting of unregulated-regulated flow transforms: Bellota

Smoothing coefficient[1] (1)	Number of iterations[2] (2)	Delta[3] (3)	Minimum threshold (1,000 cfs) (4)	Maximum threshold (1,000 cfs) (5)	Total number of data pairs (6)	MSE[4] (7)
0.2	2	0	6	52	194	10,050,441
			6	43	185	5,897,329
			43	52	9	48,302,794
			52	56	7	326,549,103
0.24			6	52	194	10,158,012
			6	43	185	6,049,790
			43	52	9	57,996,907
			52	56	7	309,817,008
Adopted transform			**6**	**52**	**—**	**10,121,872**

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to "save intermediate computations," and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 4 and 5.

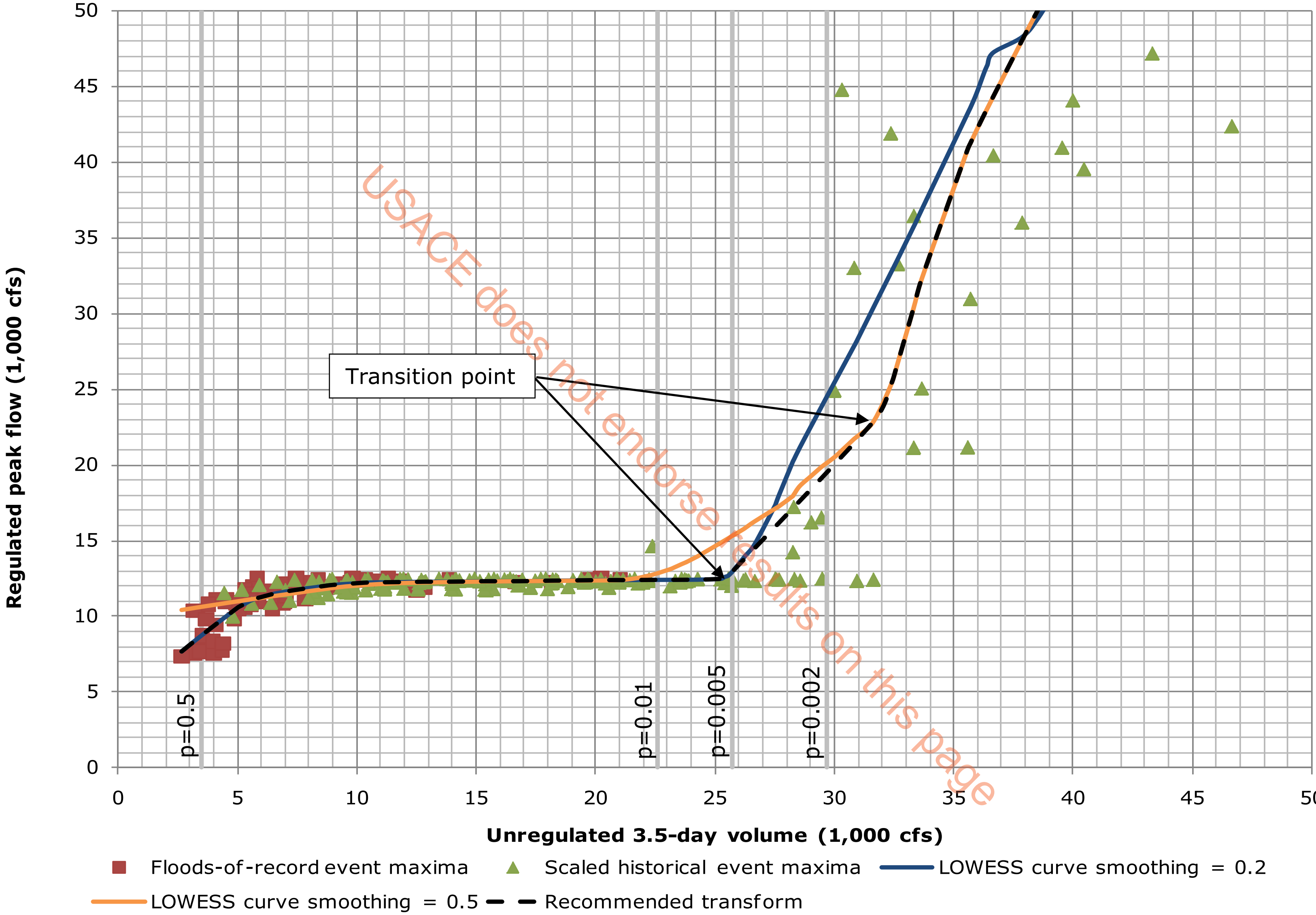

Figure 21. Unregulated-regulated flow transform and LOWESS fitted curves: New Hogan Reservoir

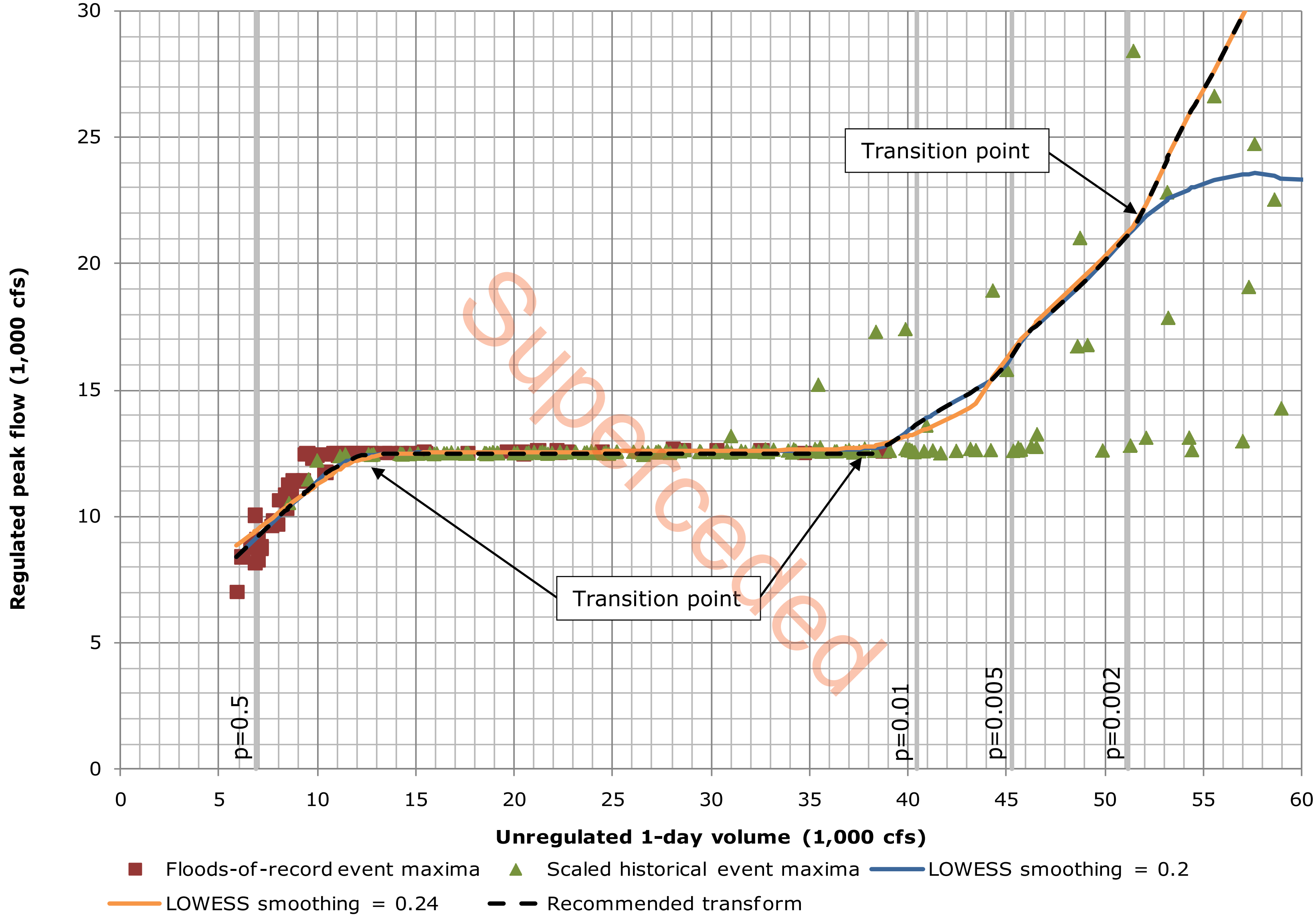

Figure 22. Unregulated-regulated flow transform and LOWESS fitted curve: Bellota

Attachment 6: Family of regulated characteristic curves

Fit the characteristic curves

We used the families of regulated characteristic curves to relate a given regulated peak flow to likely associated regulated volumes at each analysis location. We developed the families of regulated characteristic curves for New Hogan Reservoir and at Bellota by fitting transform curves through the pairs of event regulated volumes, as average flows, and regulated peak flows. The fitting is similar to how we developed the unregulated-regulated transforms detailed in Attachment 5. The datasets we used include both historical and scaled events to define the extreme ends of the flow transform curve.

We initially fitted these curves to the data pairs of historical and scaled events using the LOWESS regression technique and parameters shown in Table 28 and Table 29 for New Hogan Reservoir and at Bellota. In this initial fitting we used the entire event maxima dataset for the given analysis location. Because the flows of interest correspond to events equal or larger than the p=0.5 event, but less than or equal to the p=0.002 event, we truncated the datasets of event pairs to the minimum and maximum regulated flow thresholds specified in columns 5 and 6 of Table 28 and Table 29 for selection of the appropriate LOWESS smoothing coefficient to use in developing the characteristic curves. Highlighted in grey in Table 28 and Table 29 are the LOWESS fitted curves with smoothing coefficients listed in column 2 used in fitting the final characteristic curves for the duration specified in column 1 over the range with minimum and maximum flow thresholds specified in columns 5 and 6.

Review and adopt the characteristic curves

We reviewed and adjusted the curves initially fitted with the LOWESS procedure using the same process detailed for fitting the unregulated-regulated flow transforms. Here, the only difference is that the "break point" is defined by the downstream objective flow (12,500 cfs). Thus the mean square errors in the LOWESS fitted curves were compared over these 2 ranges for each characteristic curve.

From this review we found:

- The family of regulated characteristic curves were consistent between durations at New Hogan Reservoir. That is, they do not cross.
- The family of regulated characteristic curves we initially fitted with LOWESS were inconsistent for events with regulated peaks larger than the channel capacity constraint of 12,500 cfs. This inconsistency is a result of the effect large local flows have at this operation point. Specifically, such large peak local flows contribute to relatively high regulated peak flows for the associated regulated volumes. Therefore, the slope of the characteristic curves at Bellota is less than that seen in the characteristic curves at New Hogan Reservoir, particularly for shorter durations.
- After initially fitting the curves at Bellota, we found that the 3-day and 7-day curves crossed the 1-day curve. Therefore we set the 3-day

characteristic curve equal the 1-day curve at their initial point of intersection, and the 7-day curve equal the 3-day curve at their initial point of intersection.

- The fit of the curves at Bellota was sensitive to large peaks in local flow such as those computed directly for the 1997, 1998, and 2006 events.
- The characteristic 1-, 3, and 7-day volumes at Bellota for events with annual exceedence probabilities equal p=0.002 are less than the characteristic volume associated with New Hogan Reservoir for the same annual exceedence probability because of this effect large local flows had on the fit of the characteristic curves. However, the regulated peak flow at Bellota is always equal or larger than the peak at New Hogan Reservoir for the same exceedence probability.

Based on this review, we adopted the adjusted families of curves.

We show in Figure 23 through Figure 27 the regulated characteristic curves corresponding to New Hogan Reservoir. In addition, we include tabulations of this family of regulated characteristic curves in an MS Excel file on the DVD included with the original report.

We show in Figure 28 though Figure 32 regulated characteristic curves corresponding to Bellota. In addition, we include tabulations of this family of regulated characteristic curves in an MS Excel file on the DVD included with the original report.

In Figure 23 through Figure 32 we show the characteristic curves in black, the floods-of-record event maxima in red squares, the historical scaled event maxima in green triangles, and the initial LOWESS fitted flow curves in blue for comparison.

Table 28. LOWESS parameters for fitting the family of regulated characteristic curves and resulting errors: New Hogan Reservoir

Duration (days) (1)	Smoothing coefficient[1] (2)	Number of iterations[2] (3)	Delta[3] (4)	Minimum threshold (1,000 cfs) (5)	Maximum threshold (1,000 cfs) (6)	Total number of data pairs (7)	LOWESS curve MSE[4] (8)	Characteristic curve MSE (9)
1	0.2	2	0	8	22	201	285,737	295,465
3	0.7						1,833,013	1,995,342
7	0.2						4,004,463	4,767,174
15	0.2						3,939,439	6,168,764
30	0.2						2,420,500	3,930,845

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to "save intermediate computations," and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 5 and 6.

Table 29. LOWESS parameters for fitting the family of regulated characteristic curve and resulting errors: Bellota

Duration (days) (1)	Smoothing coefficient[1] (2)	Number of iterations[2] (3)	Delta[3] (4)	Minimum threshold (1,000 cfs) (5)	Maximum threshold (1,000 cfs) (6)	Total number of data pairs (7)	LOWESS curve MSE[4] (8)	Characteristic curve MSE (9)
1	0.7	2	0	8	22	201	510,466	552,352
	0.2			8	13	181	299,883	
	0.7			13	22	20	2,555,554	
3	0.2			8	22	201	1,367,756	1,806,174
	0.2			8	13	181	1,168,187	
	0.7			13	22	20	2,802,423	
7	0.2			8	22	201	2,417,325	7,982,243
15	0.2			8	22	201	3,293,534	21,812,221
30	0.2			8	22	201	2,083,062	19,331,298

Notes:

1. The fraction of points used to calculate each point of the flow transform.
2. The number of iterations used in calculating the robust fitted curve. A value of 2 returns a robust fit.
3. Delta is a nonnegative value used by the program we used to compute the LOWESS algorithm to "save intermediate computations," and reduces computation time for large datasets. In this study the datasets are small, and thus this was set to 0.
4. Mean square error over the range of interest defined by the minimum and maximum thresholds listed in columns 5 and 6.

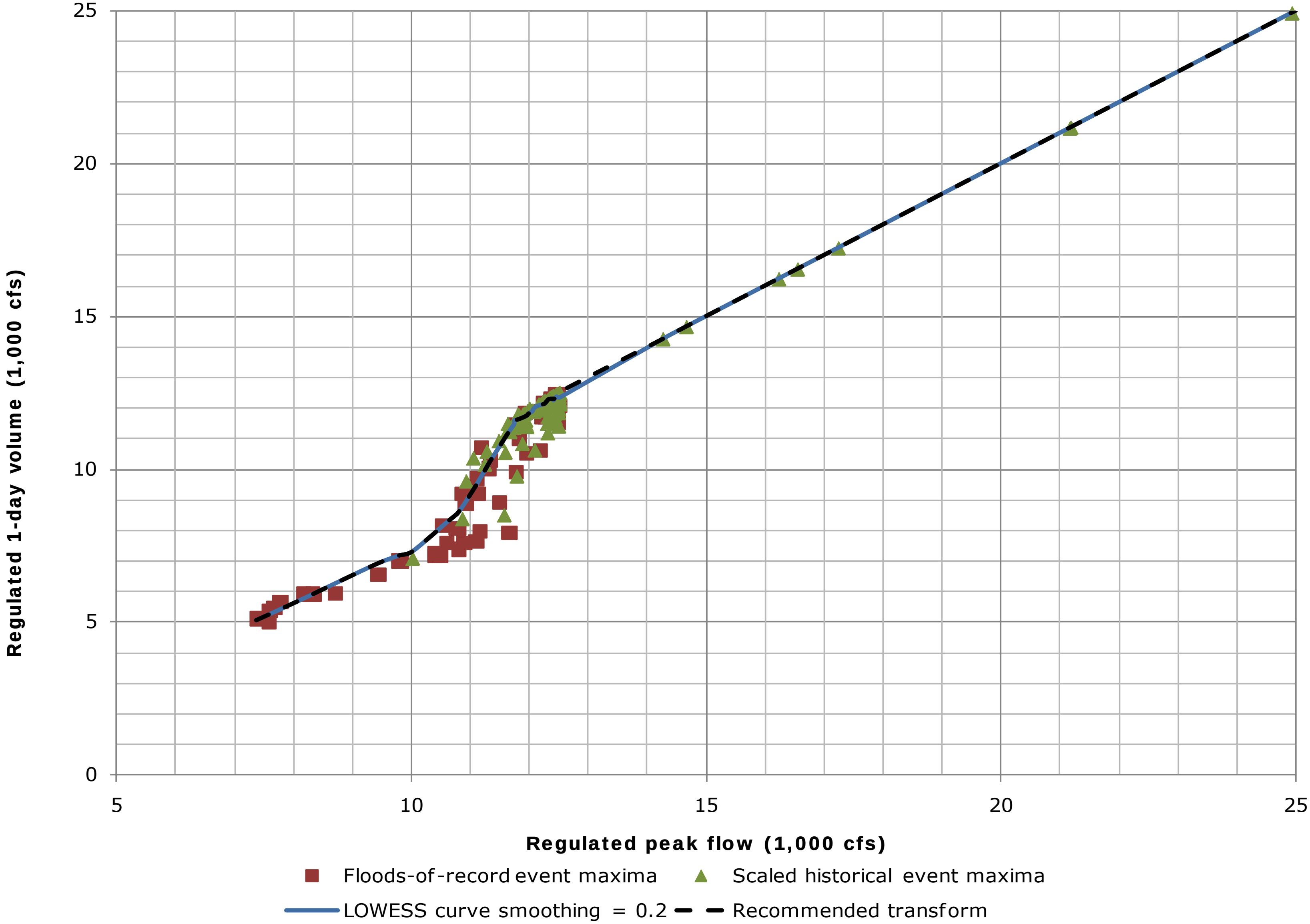

Figure 23. New Hogan Reservoir regulated characteristic curve: 1-day duration

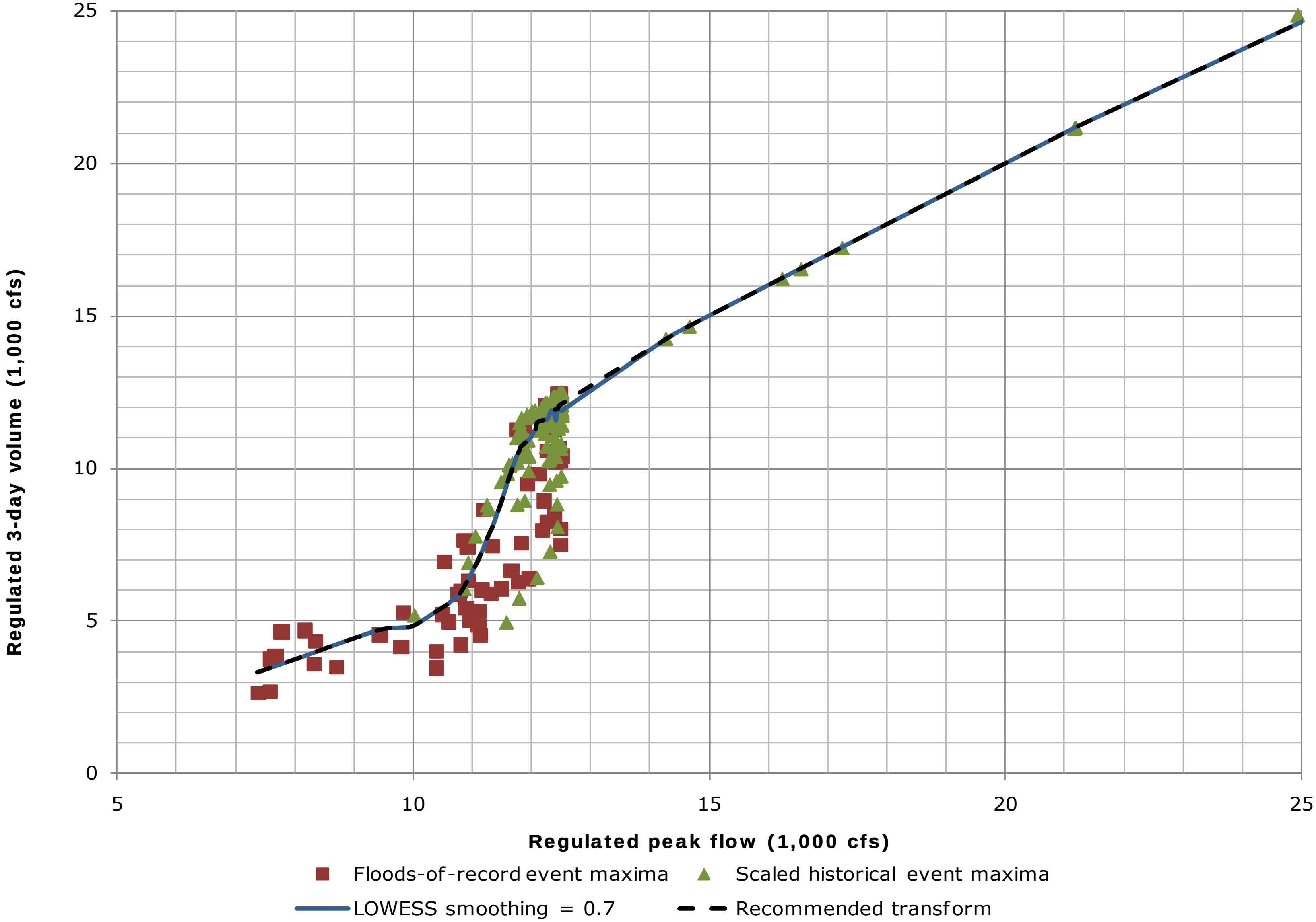

Figure 24. New Hogan Reservoir regulated characteristic curve: 3-day duration

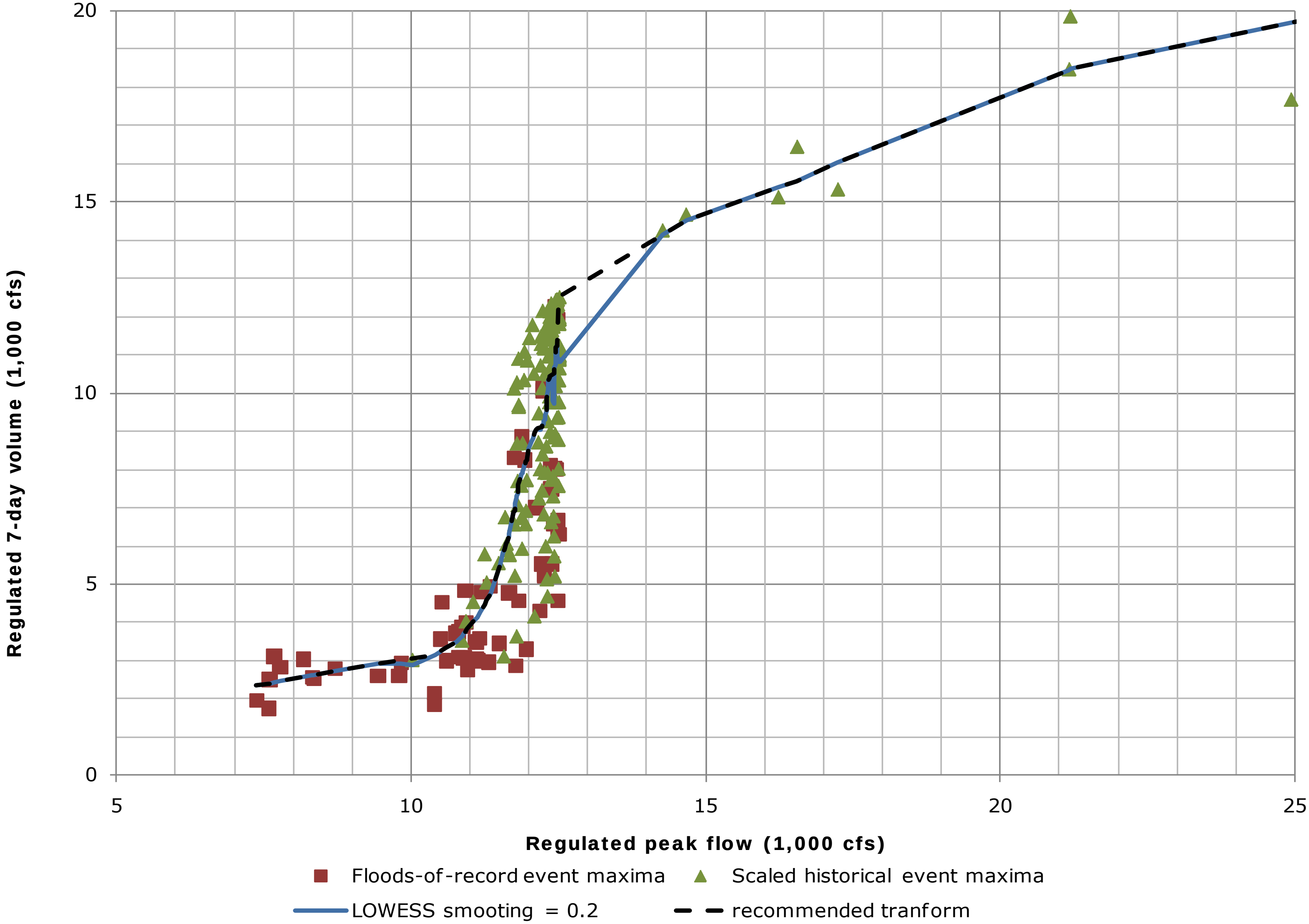

Figure 25. New Hogan Reservoir regulated characteristic curve: 7-day duration

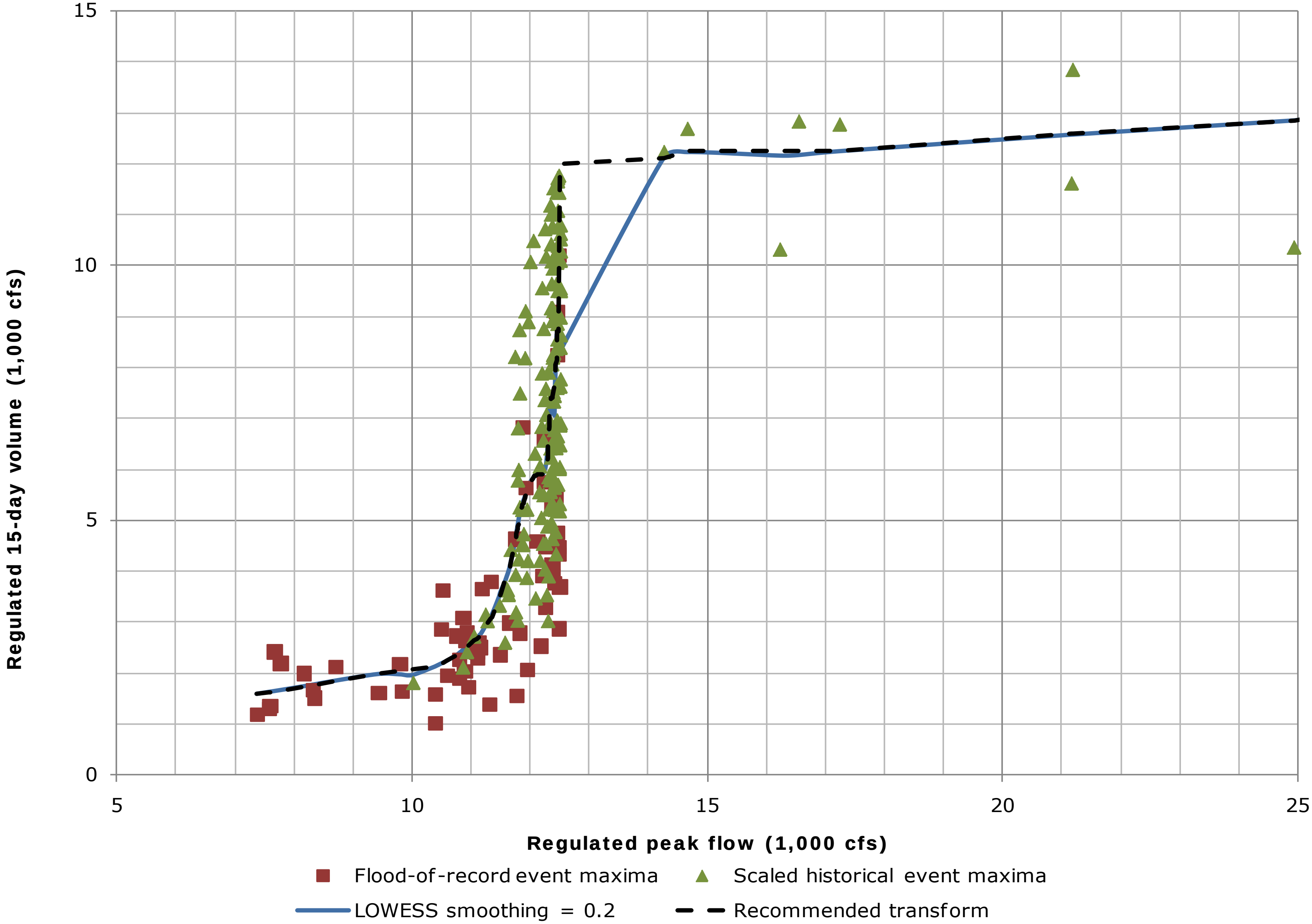

Figure 26. New Hogan Reservoir regulated characteristic curve: 15-day duration

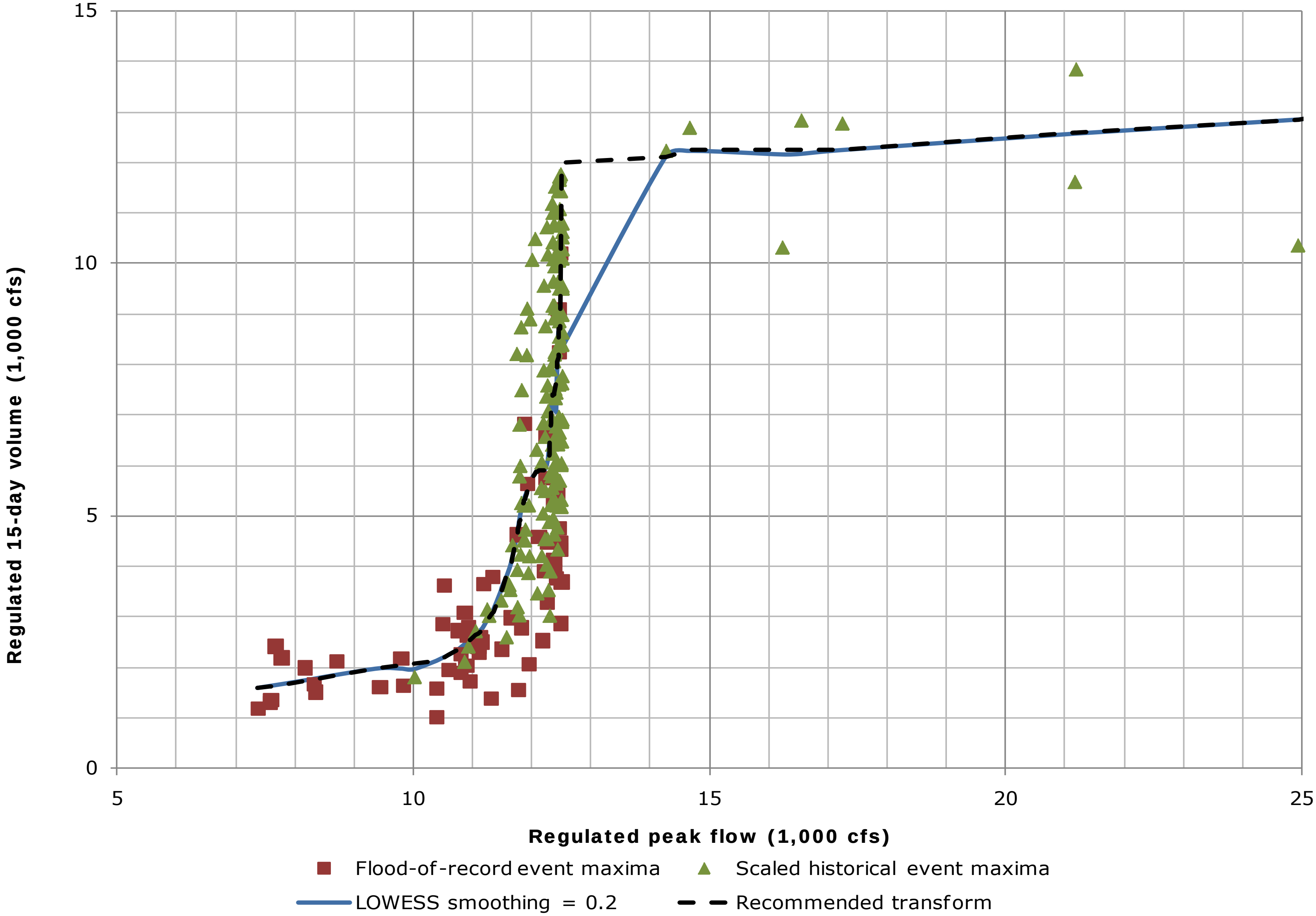

Figure 27. New Hogan Reservoir regulated characteristic curve: 30-day duration

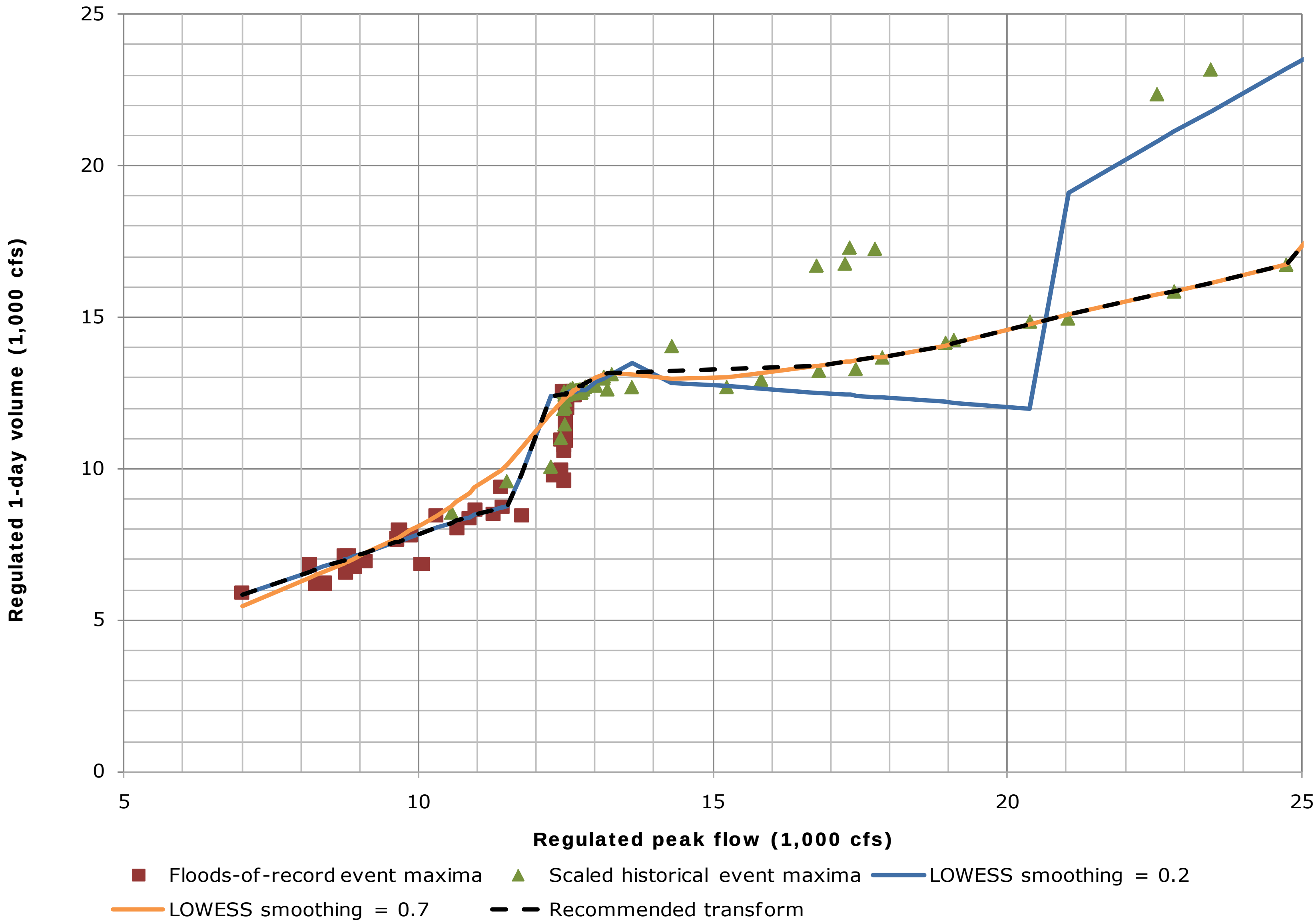

Figure 28. Calaveras River at Bellota regulated characteristic curve: 1-day duration

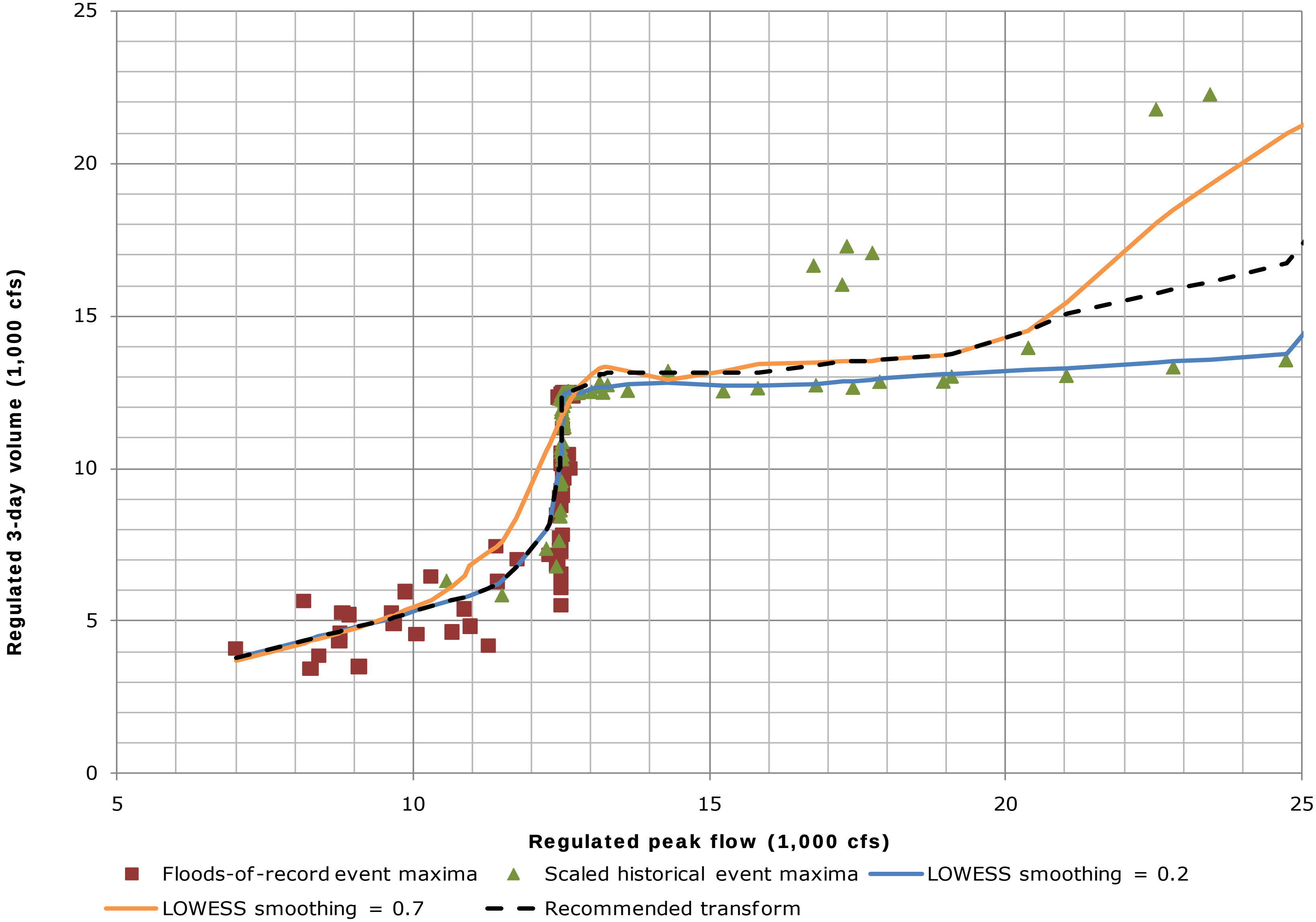

Figure 29. Calaveras River at Bellota regulated characteristic curve: 3-day duration

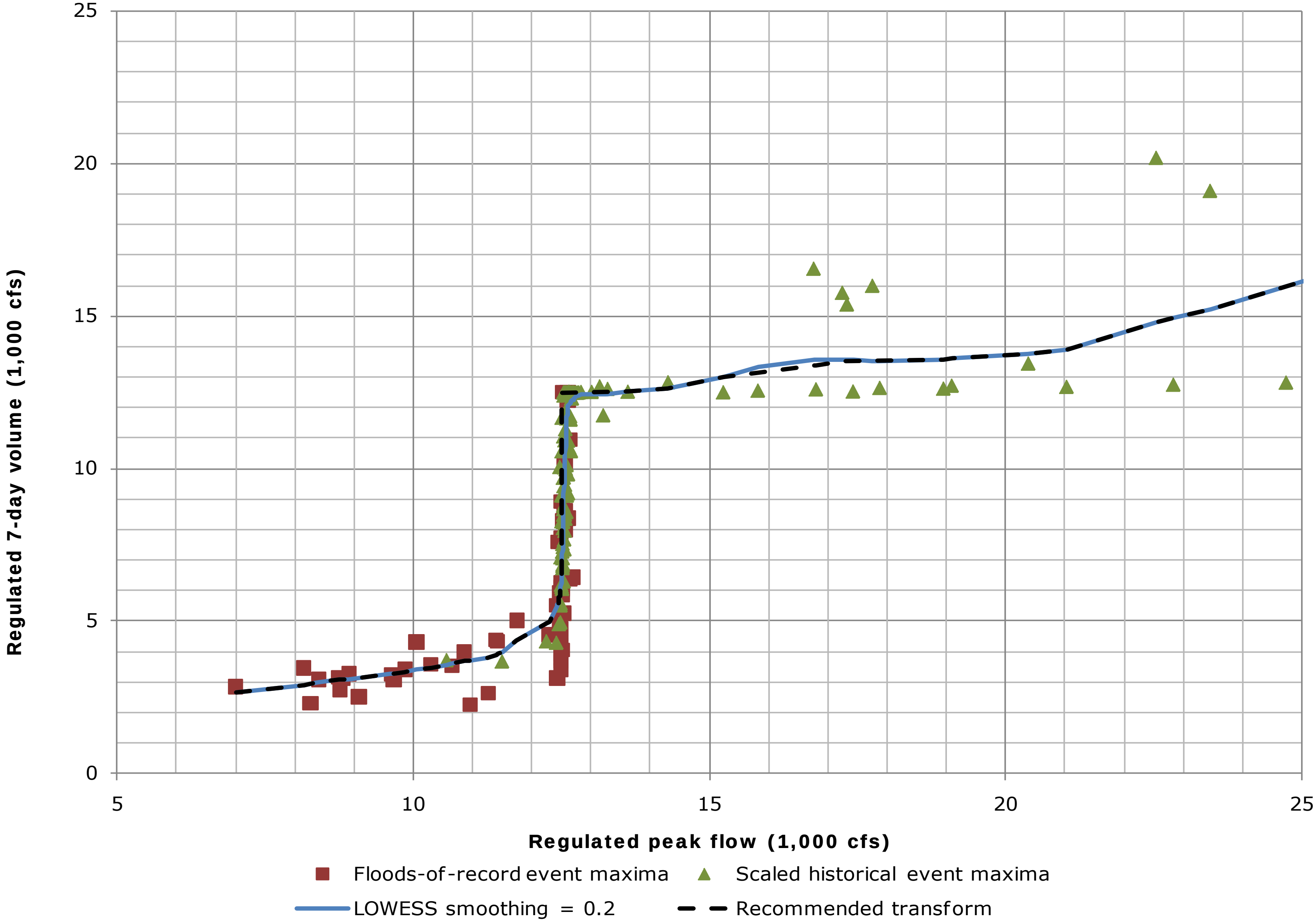

Figure 30. Calaveras River at Bellota regulated characteristic curve: 7-day duration

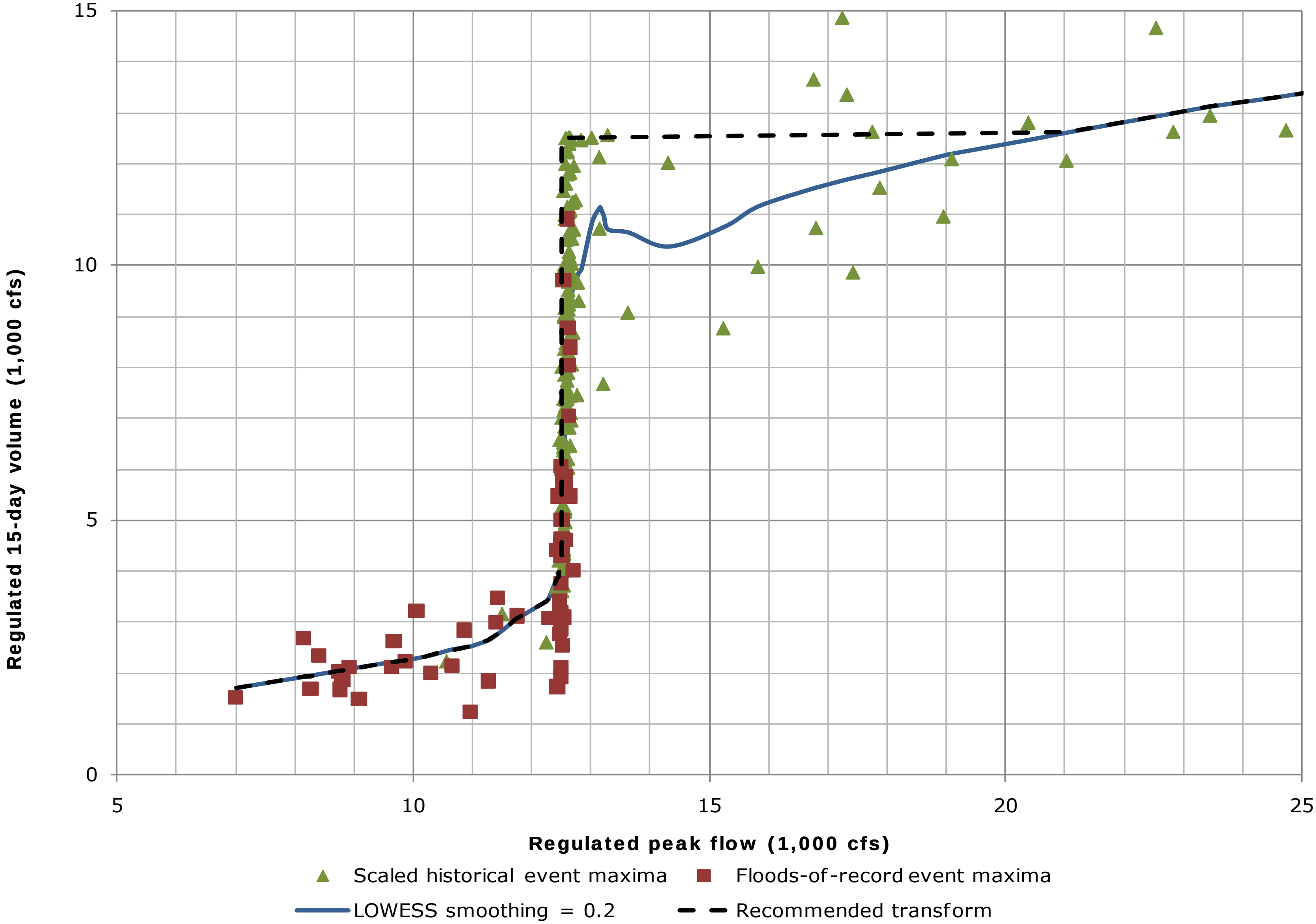

Figure 31. Calaveras River at Bellota regulated characteristic curve: 15-day duration

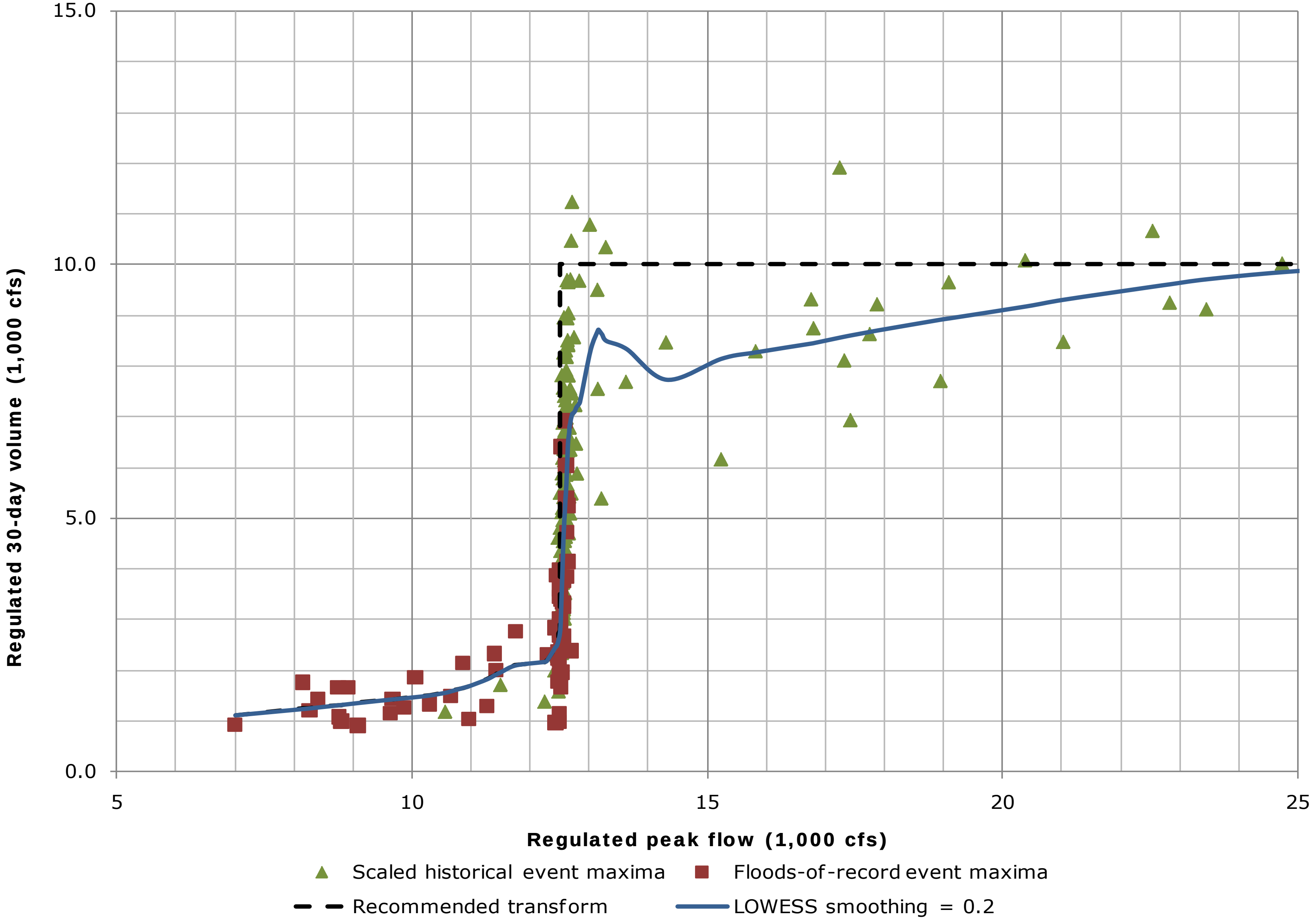

Figure 32. Calaveras River at Bellota regulated characteristic curve: 30-day duration

Attachment 7: Quality control certification

David Ford Consulting Engineers, Inc. completed Task 3, development of flow frequency curves, expected hydrographs, and documentation of procedures for contract W91238-09-D-0004—Lower San Joaquin River Feasibility Study, San Joaquin County, CA including Stockton City and nearby communities.

Notice is hereby given that all quality control activities of the technical memorandum prepared by the firm have been completed, appropriate to the level of risk and complexity inherent in the project, as defined in the Quality Control Plan. Compliance with established policy principles and procedures, utilizing justified and valid assumptions, was verified. This includes review of assumptions; methods, procedures, and material used in the analyses; the appropriateness of data used and level of data obtained; and reasonableness of the results, including whether the product is consistent with law and existing Corps policy.

REGISTERED PROFESSIONAL ENGINEER
DAVID THURMAN FORD
No. 33889
Exp. 6/30/2012
CIVIL
STATE OF CALIFORNIA

3/25/2011

David T. Ford, PhD, PE, D.WRE (date)
President
David Ford Consulting Engineers, Inc.

Appendix 1- Attachment 2

Lower San Joaquin Feasibility Study Alternative Analysis for Calaveras River at New Hogan Dam

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MEMORANDUM

To: John High and Steve Holmstrom, PE

From: Nathan Pingel, PE; Teresa Bowen, PE; and Michael Konieczki, PE

Date: August 12, 2011

Subject: Contract W91238-09-D-0004-0004 modification 2: Lower San Joaquin River feasibility study, San Joaquin County, CA, including Stockton City and nearby communities

Deliverable for task 6 and option task 1: Use existing New Hogan Dam HEC-ResSim model to evaluate re-operation alternatives to achieve 200-yr protection downstream and investigate the impact of downstream channel improvements to achieve 200-year protection downstream

Situation

In support of the lower San Joaquin River feasibility study (LSJR FS), we completed a hydrologic analysis of the Calaveras River, specifically focusing on analysis points at New Hogan Reservoir and Bellota. The results of this analysis are described in our June 20, 2011, report, *Lower San Joaquin River feasibility study: Calaveras River frequency analysis and hydrographs.* In that report, we presented unregulated flow-frequency curves and unregulated to regulated flow transforms for the analysis points noted above. Using these 2 products, we also presented regulated peak flow-frequency curves at the analysis point locations. This work was completed for the Sacramento District of the US Army Corps of Engineers (Corps).

Table 1 summarizes the peak regulated flow at 2 locations along the Calaveras River—immediately below New Hogan Dam, and at the downstream reservoir operating point of Bellota—for 2 events: the p=0.005 event and the p=0.01 event.

The reservoir is operated to limit the flow at Bellota to 12,500 cfs, unless a larger release is required by the reservoir operation rules or the available flood storage in the dam is exhausted. The downstream peak flow at Bellota is a function both of reservoir releases and local uncontrolled flow from the watershed area between New Hogan and Bellota.

As part of the LSJR FS, the Corps and the local sponsor, the San Joaquin Area Flood Control Agency (SJAFCA), are evaluating alternative flood risk reduction measures that will provide greater flood protection. The focus of these alternatives is to protect downstream areas from flooding from events more common than the p=0.005 event.

Table 1. Peak regulated flow for selected annual exceedence probabilities[1]

Annual exceedence probability (1)	Peak regulated flow below New Hogan Dam (cfs) (2)	Peak regulated flow at Bellota (cfs) (3)
0.01	12,367	13,634
0.005	12,903	16,409

Notes:
1. Values are as reported in the June 2011 report *Lower San Joaquin River feasibility study: Calaveras River frequency analysis and hydrographs*

Tasks

Our task is to use the baseline hydrologic analysis as documented in our June 2011 report and evaluate 2 alternatives:

1. Modifications to New Hogan Reservoir to reduce p=0.005 peak flows downstream to 12,500 cfs.
2. Modifications to the downstream channel capacity to contain the p=0.005 peak flows. (Alternative channel capacities under consideration by the project team include increases from 12,500 cfs to 15,000 cfs, 18,000 cfs, or 21,000 cfs.)

This evaluation is from a hydrologic perspective only and to support initial alternative screening. This evaluation does not include the assessment of risk reduction, as measured with reduction in expected annual damage, nor does it include an explicit consideration of uncertainty in of the assessments of "level of protection" or ability of the system to pass or control an event of specified probability.

Actions

To evaluate the 2 alternatives above, we:

1. Prepared an exposition of the reservoir simulation results for selected events from our June 2011 report, which allowed us to elaborate specifically on whether downstream channel capacity was exceeded, and if so, why. Doing so allows us to focus on the predominant factors influencing flooding downstream of New Hogan:
   - The inflow to New Hogan Reservoir.
   - The local uncontrolled flow between New Hogan and Bellota.
   - The use of the flood storage in New Hogan Reservoir.
   - The rate-of-change reservoir operating rule and the emergency spillway release diagram (ESRD) minimum releases.

   The events selected for this exposition are those that have peak flows approximately equal to the p=0.005 flow at Bellota. These events are described in Attachment A.
2. Evaluated the coincident probabilities of New Hogan Reservoir inflows to probability of local uncontrolled flow. Like the exposition of the reservoir simulation results, the evaluation of coincident probabilities informs the assessment of alternative measures that could reduce the downstream

regulated peak flow. In addition, this evaluation provides guidance for critical storm centering for the rainfall-runoff portion of the overall LSJR FS.

For this evaluation, we completed a flow-frequency analysis on the local flow time series used in the baseline analysis. Using that limited-use local flow-frequency curve and the events described in step 1, we assessed the coincident probabilities between the reservoir inflow and the local flow hydrographs. [The flow-frequency curve developed for this step is intended only for this purpose and not intended to be adopted as a study product, thus referred to as a "limited-use local flow-frequency curve." The study product is being developed through rainfall-runoff model simulations of design storms.]

This analysis is described in Attachment B.

3. Developed and evaluated design events to assess further the sensitivity of reservoir storage and uncontrolled local flows to the peak regulated flow at Bellota. Design events (or hydrographs) are historical events scaled to a specific peak and/or volume(s) of specified probability.

   We developed design events focused at p=0.005 flow at New Hogan and Bellota. These design events are based on historical events and scaled using consistent methodology as in the baseline analysis. We also developed and simulated design (scaled) events for the p=0.01 and p=0.002 flows at both locations.

   This analysis is described in Attachment C.

4. Evaluated the impact of increased flood control storage in New Hogan Reservoir using selected events from our June 2011 report and the results from the actions noted above. Specifically, we focused here on whether increased flood control storage could reduce peak flows at Bellota. These selected events are the same as in step 1 above.

   The analysis plan is included in Attachment D and the analysis is described in Attachment E.

5. Evaluated the impact of increased channel capacity between New Hogan Reservoir and Bellota. This increased channel capacity allows for conveyance of both uncontrolled local flows and reservoir releases.

   This is described in Attachment F.

Findings

From the analysis described above and review of the baseline hydrologic analysis, we found:

- Peak regulated flows at Bellota are a result of both the uncontrolled local flow between New Hogan and Bellota and New Hogan Reservoir releases. New Hogan releases are determined by the prescribed flood control storage, the reservoir inflow, and the dictated reservoir operation rules. So, capacity exceedence at downstream locations may be caused by excessive local flow, excessive reservoir release, or both.
- The probability of the reservoir inflow and the coincident local flow varies by event. A predictable relationship does not exist. For some historical

events, the local flow is rarer than the reservoir inflows. And for others, the opposite is true.

- The p=0.005 3-day volume from the New Hogan frequency curve is less than the dedicated flood storage at New Hogan Reservoir.
- The p=0.005 4-day volume from the New Hogan frequency curve is greater than the dedicated flood storage at New Hogan Reservoir. For actual simulations of design (scaled) events, which include reservoir releases, the total required stored volume does not exceed the dedicated storage for the 1958, 1986, 1997, and 2006 design pattern events. [For the 1998 design pattern event, the stored volume exceeds the dedicated flood storage. However, to scale the 1998 event to the design criterion requires a scaling factor larger than that recommended in Corps' EM 1110-2-1415 (USACE 1993).]
- For the evaluation of selected events, in most cases, the local flow alone exceeded the downstream channel capacity. For the event where local flow did not exceed the channel capacity, the 1958 event scaled by 1.4, a minimum of 14,160 ac-ft of additional storage is needed to maintain a flow at Bellota below 12,500 cfs.

Results

Based on our findings, additional storage alone in New Hogan Reservoir will not reduce the p=0.005 event flow to less than or equal to 12,500 cfs at Bellota. Increased storage may reduce the regulated peak flow-frequency curve, but it will not lower it below the peak local flow-frequency curve for the watershed area between the dam and Bellota.

To "contain" the p=0.005 flow, increased channel capacity is required. As a minimum, for the current watershed condition, the increased channel capacity would need to be equal to or greater than the peak p=0.005 local flow from the watershed area between the dam and Bellota. An alternative to increased channel capacity would be to reduce the peak local flow-frequency curve.

The limited-use peak local flow-frequency curve presented herein is for this analysis only. As a part of the LSJR FS, a separate effort is being completed to develop a local flow-frequency curve using rainfall-runoff models and design storms. The results of that analysis were not available for use here. Once that analysis is completed and adopted, the impact of a revised peak local flow-frequency curve to the conclusions presented herein should be considered.

Guide to attachments

As described above, the attachments summarize the analysis completed to answer the questions posed. Below is the table of contents for these attachments:

References

Cohn, Tim. (2007). PeakfqSA, version 0.937 [Software]. <http://www.timcohn.com/TAC_Software/PeakfqSA/>.

Interagency Advisory Committee on Water Data (IACWD). (1982). *Guidelines for determining flood flow frequency, Bulletin 17B.* US Geological Survey, Reston, VA.

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USACE. (2004). *Farmington Dam and Reservoir, Littlejohn Creek, California, Water control manual, Appendix IV to Master water control manual, San Joaquin River Basin, California,* Sacramento District, Sacramento, CA.

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USACE. (1983). *New Hogan Dam and Lake, Calaveras River, California, Water control manual, Appendix III to Master water control manual, San Joaquin River Basin, California,* Sacramento District, Sacramento, CA.

US Geological Survey (USGS). (2011). *Regional skew for California, and flood frequency for selected sites in the Sacramento–San Joaquin River Basin, based on data through water year 2006: U.S. Geological Survey Scientific Investigations Report 2010–5260*, USGS, Sacramento, CA.

USGS. (forthcoming). Regional Skews for 1-Day, 3-Day, 7-Day, 15-Day, and 30-Day Duration Discharge for the Central Valley Region of California, USGS, Sacramento, CA.

Attachment A. Exposition of selected reservoir simulations from baseline analysis

Overview

For the analysis in our June 2011 report, we routed 60 historical events and 190 scaled-versions of the historical events (19 events times 10 scale factors each) through the reservoir simulation model. Computer program HEC-ResSim was used to develop the New Hogan Reservoir model and to complete the simulations. In that report, the results of the simulations were summarized in the unregulated to regulated flow and regulated flow to regulated volume transform plots; each point in the figures represented a reservoir simulation of a historical or scaled historical event.

As a part of this current analysis, and to support ongoing discussions of the baseline analysis described in the June 2011 report, we include here an exposition of a subset of the reservoir simulations completed.

Selection of events

We selected 8 events used in the baseline hydrologic analysis that represent approximately a p=0.005 regulated peak flow at Bellota. (An event with a regulated peak flow at Bellota equal to the p=0.005 event does not necessarily correspond to an event with a New Hogan Reservoir inflow equal to the p=0.005 event.) The regulated peak flow at Bellota for the p=0.005 event is 16,407 cfs, per the June 2011 report. Selected events are shown in Table 2. Column 1 of Table 2 notes the selected historical event; the associated start and end dates are listed in columns 3 and 4. Column 2 notes if the event was a scaled version of this historical event or not; the value indicates the factor that was used to scale uniformly the historical event. For reference, column 5 notes the peak regulated flow at Bellota from the reservoir operation simulations and column 6 indicates the peak local flow used as input for the simulation. In the following section, the reservoir simulations are further described in graphical form.

For reference, Figure 1 shows the Bellota unregulated to regulated flow transform from the June 2011 analysis with these selected events labeled. For the development of that transform, the 2006 event was not included, but has been added to the figure for reference purposes.

Reservoir operation simulation for selected events

Reservoir simulation routings for each of the events listed in Table 2 are shown in Figure 2 through Figure 9. For each figure, we include a plot showing the water surface elevation at New Hogan, inflow, outflow, local flow between New Hogan and Bellota, unregulated flow at Bellota (flow that would have occurred with no upstream reservoir), and regulated flow at Bellota (local flow plus reservoir releases).

Critique of simulations and events

Table 3 summarizes the selected event simulations. In column 3 of Table 3 we note whether or not the downstream channel capacity of 12,500 cfs was exceeded. If it was, we note in column 4 the prominent factor from the simulation that caused that to occur. In column 5 we provide notes about mitigation alternative(s) (additional flood storage, revision to the ESRD, or lowering the flood pool) that may be considered to lessen the peak flow downstream. And, in column 6, we note what the resulting downstream peak flow for that event could be with those mitigation alternative(s) in place. This list of alternatives is for planning purposes only and is not the result of a full alternative analysis.

Table 2. Selected historical and scaled historical events

Event (1)	Scale factor (2)	Start date (3)	End date (4)	Peak regulated flow at Bellota[1] (cfs) (5)	Peak local flow[1,2] (cfs) (6)
1907	2.2	Mar 1, 1907	Apr 13, 1907	16,543	13,195
1958	1.4	Mar 10, 1958	Apr 29, 1958	16,759	3,070
1969	3.0	Jan 3, 1969	Mar 1, 1969	12,500[3]	4,777
1986	1.6	Jan 21, 1986	Mar 31, 1986	12,500[3]	9,359
1997	2.2	Dec 1, 1996	Feb 15, 1997	15,822	14,714
1998	1.6	Jan 1, 1998	Mar 15, 1998	15,906	15,098
1999	1.0	Feb 6, 1999	Feb 12, 1999	12,500[3]	5,620
2006	1.6	Mar 24, 2006	Apr 24, 2006	12,500[3]	11,698

Notes:
1. Peak regulated flow and peak local flow values are not necessarily coincident in time.
2. Local flow is the uncontrolled watershed contribution from New Hogan Dam to Bellota.
3. Reservoir releases adjusted to 12,500 cfs from HEC-ResSim computed releases to compensate for known routing issues in the computer program. For these simulations, sufficient flood storage is available for the event.

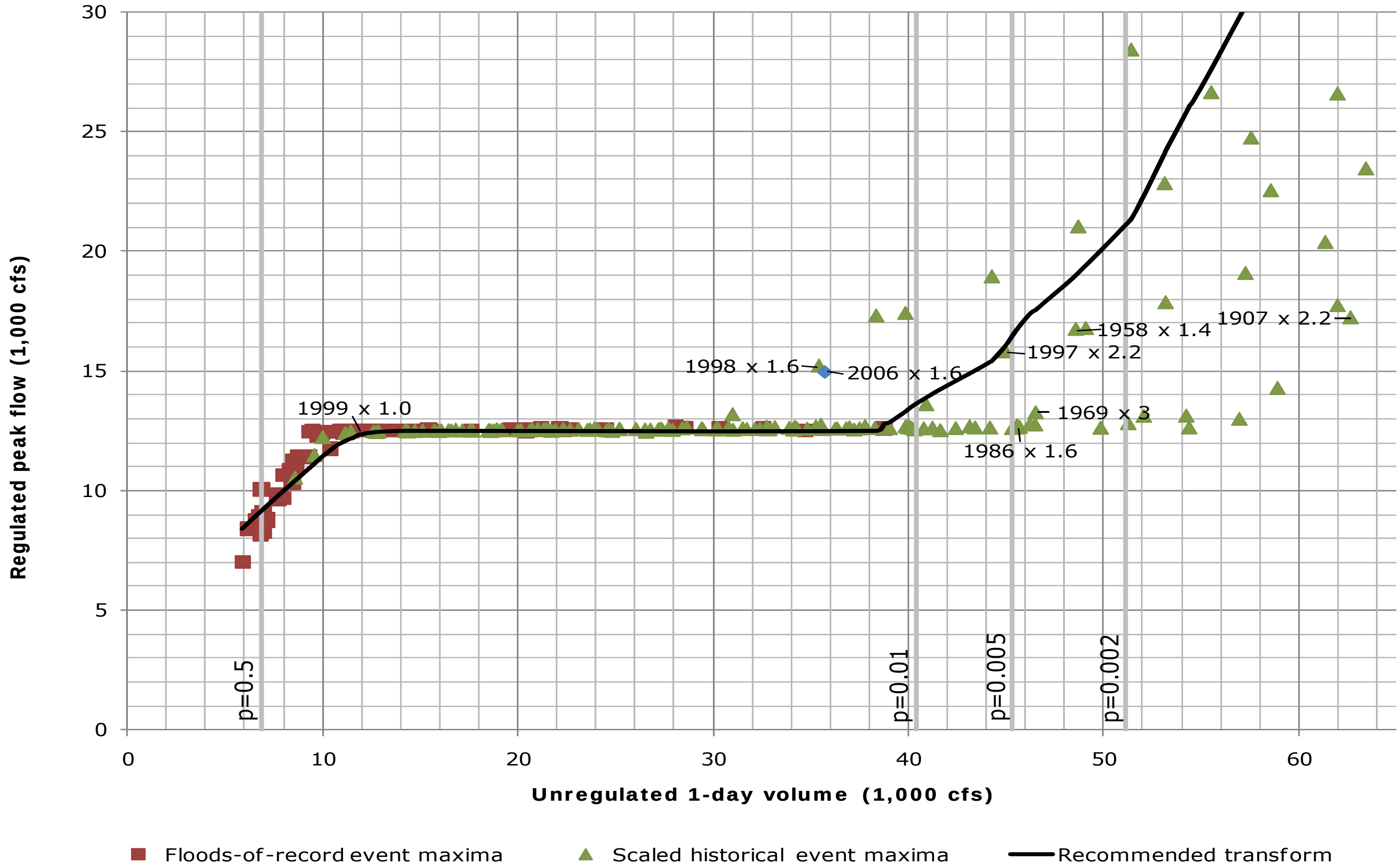


Figure 1. Unregulated to regulated flow transform from June 2011 baseline analysis: Calaveras River at Bellota with highlighted selected events. The 2006 event shown with a blue diamond was not used for flow transform development.

Table 3. Critique of controlling factor for simulations of selected historical and scaled historical events

Event (1)	Scale factor (2)	Channel capacity at Bellota exceeded? (3)	If channel capacity at Bellota is exceeded, why? (4)	Notes about possible New Hogan mitigation alternative(s) (5)	Peak flow at Bellota after modification (cfs) (6)
1907	2.2	Yes	Local flows	Additional flood storage will not keep flow at Bellota < 12,500 cfs	N/A
1958	1.4	Yes	ESRD release	1. Remove or revise ESRD 2. Lower flood pool to 661 ft[2]	12,500[1] 12,500[1]
1969	3.0	No	N/A	—	—
1986	1.6	No	N/A	—	—
1997	2.2	Yes	Local flows	Additional flood storage will not keep flow at Bellota<12,500 cfs	N/A
1998	1.6	Yes	Local flows	Additional flood storage will not keep flow at Bellota<12,500 cfs	N/A
1999	1.0	No	N/A	—	—
2006	1.6	No	N/A	—	12,500[1]

Notes:
1. Reservoir releases adjusted to 12,500 cfs from HEC-ResSim computed releases to compensate for known routing issues in the computer program. For these simulations, sufficient flood storage is available for the event.
2. A lowered flood pool to elevation 661 ft translates to additional flood storage of 14,157 ac-ft.

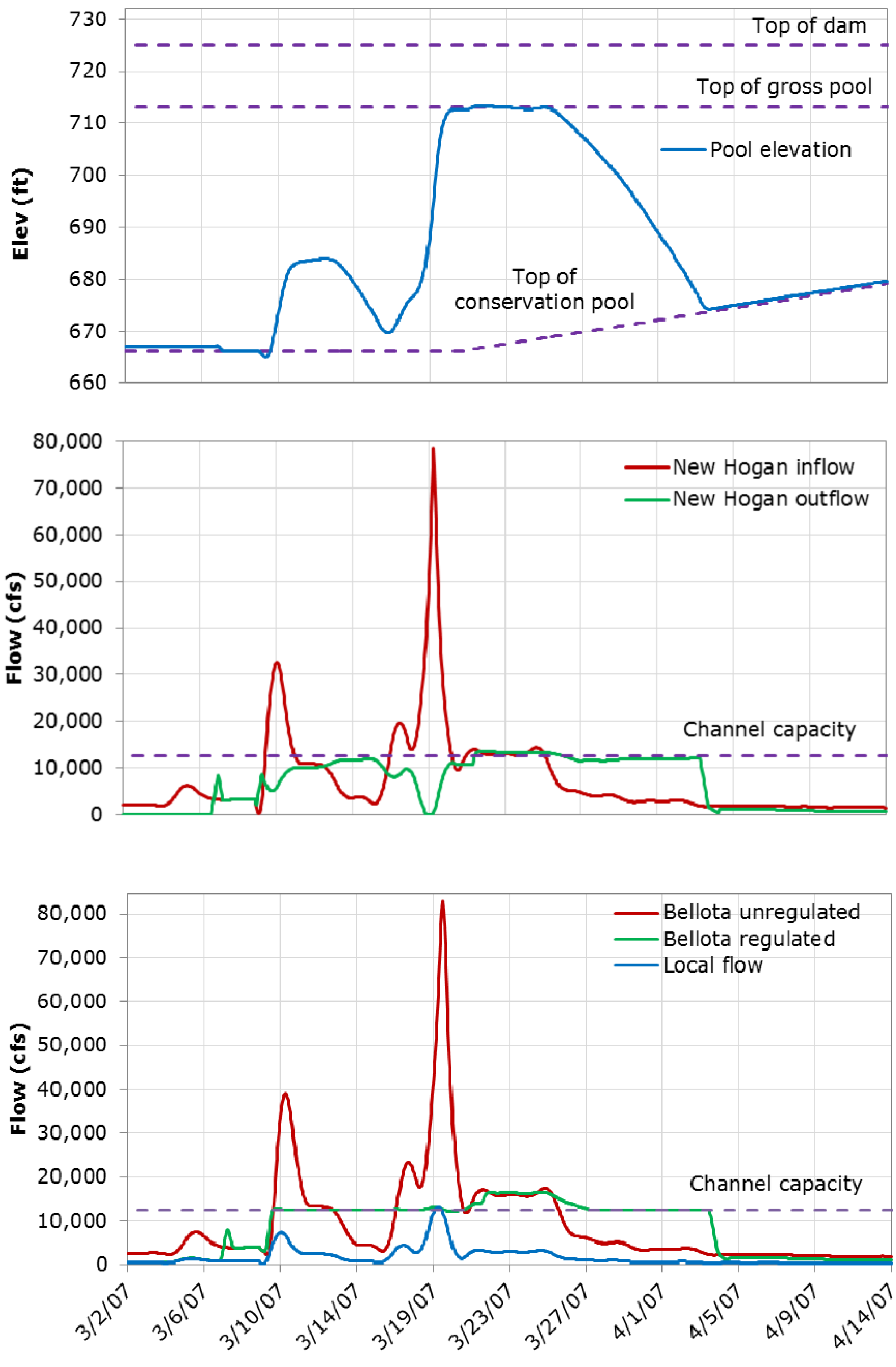


Figure 2. New Hogan routing of 1907 event scaled by 2.2

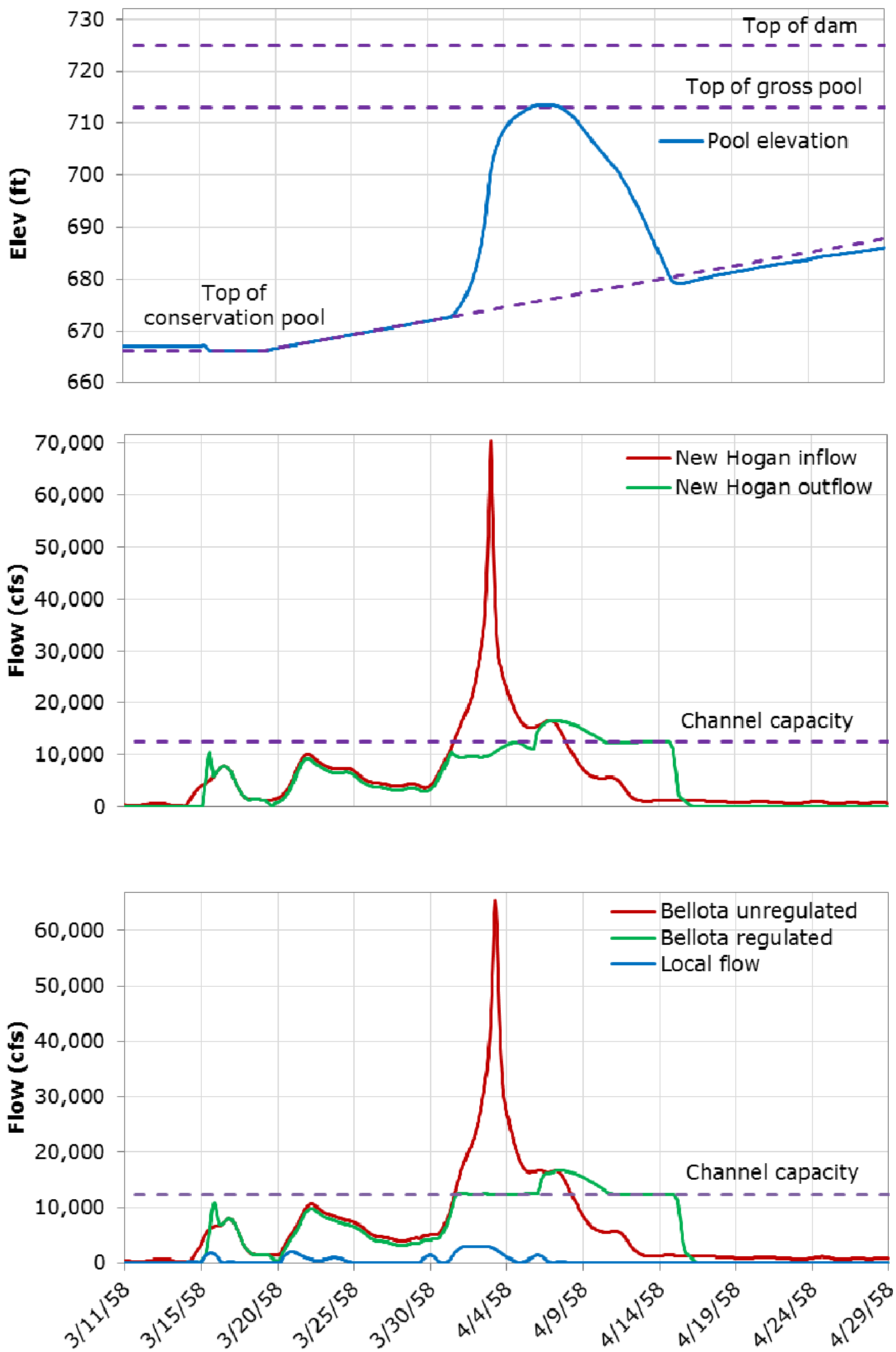


Figure 3. New Hogan Reservoir routing of 1958 event scaled by 1.4

Figure 4. New Hogan Reservoir routing of 1969 event scaled by 3.0

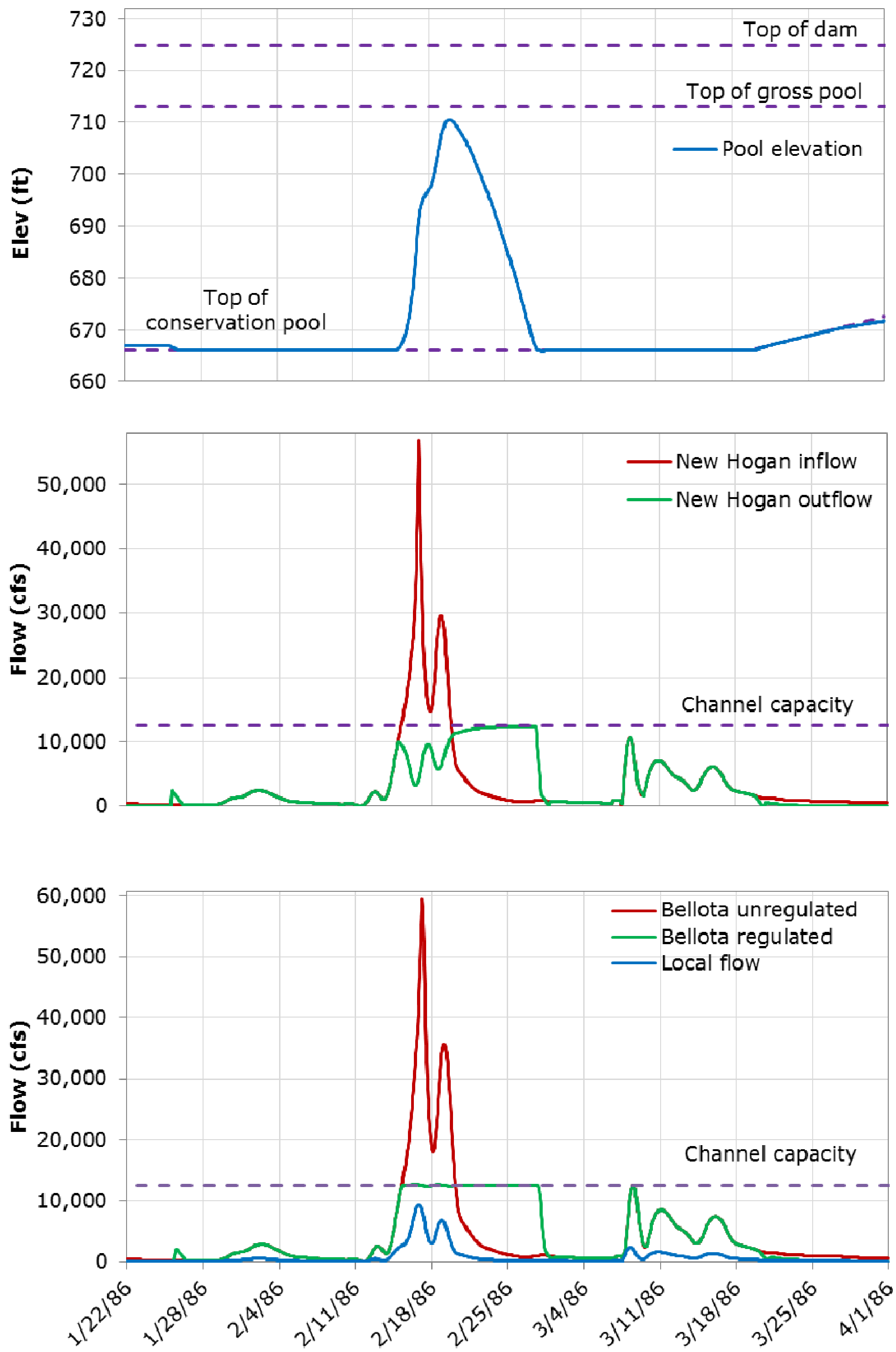


Figure 5. New Hogan Reservoir routing of 1986 event scaled by 1.6

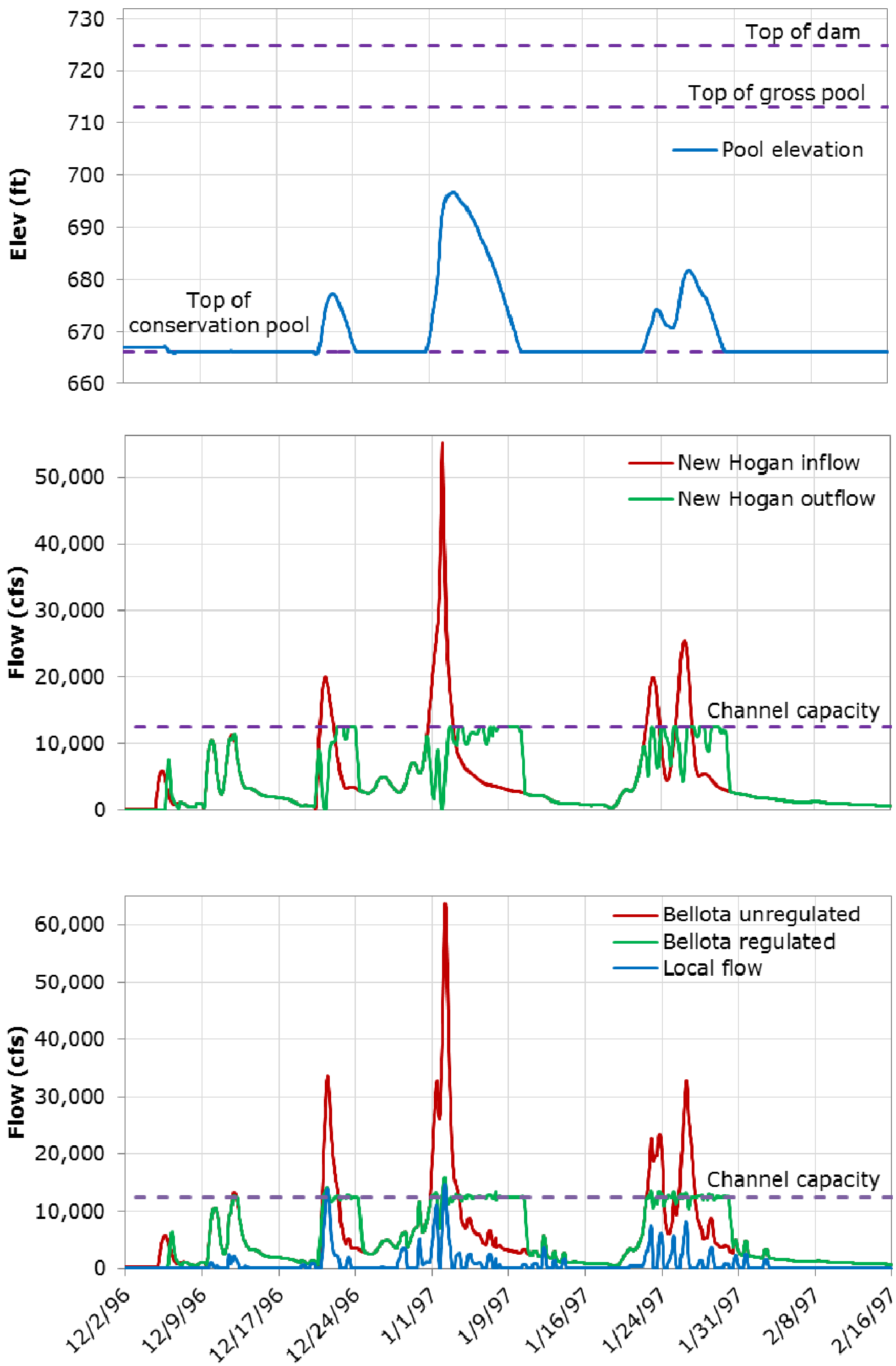


Figure 6. New Hogan Reservoir routing of 1997 event scaled by 2.2

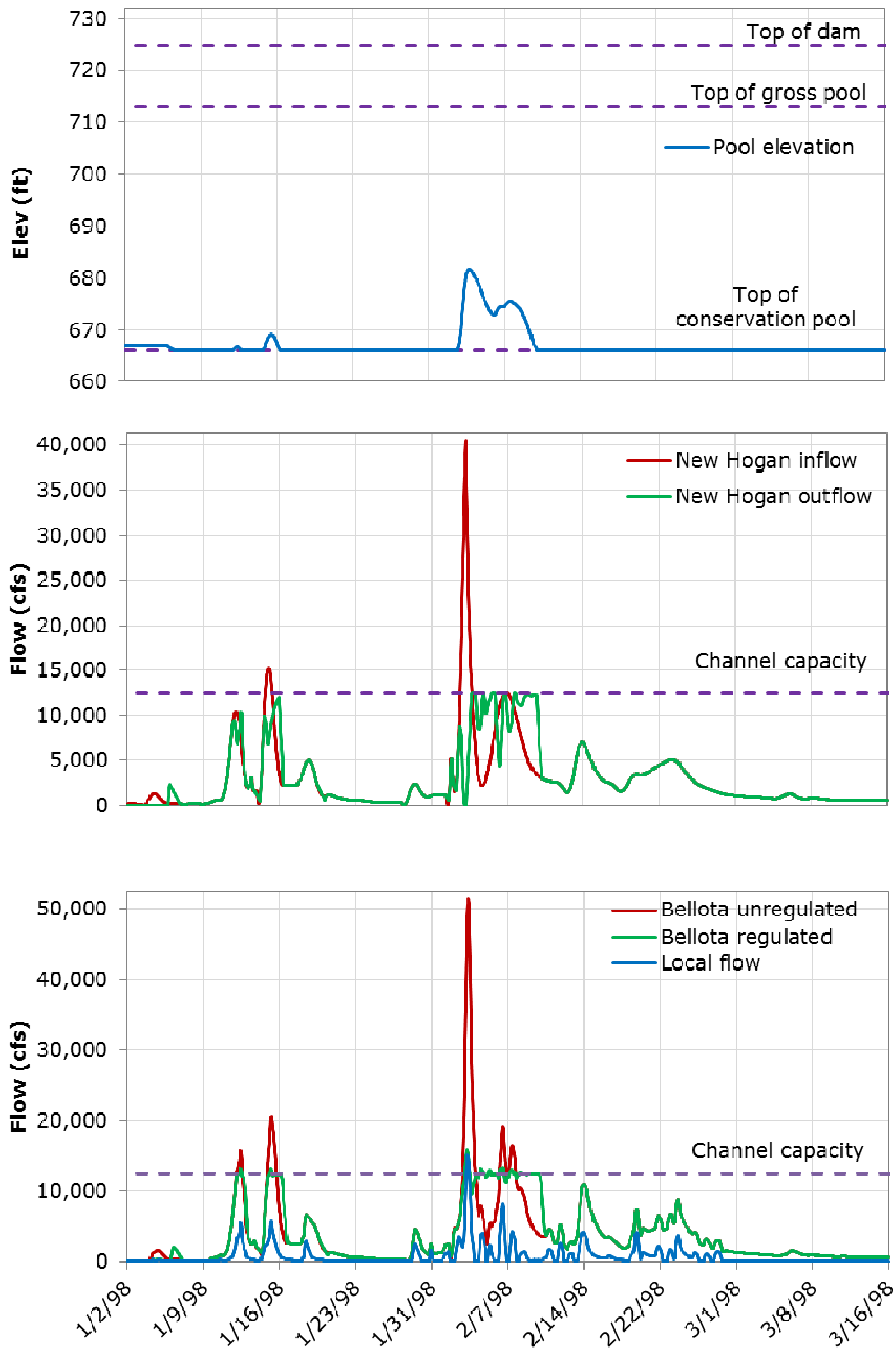


Figure 7. New Hogan Reservoir routing of 1998 event scaled by 1.6

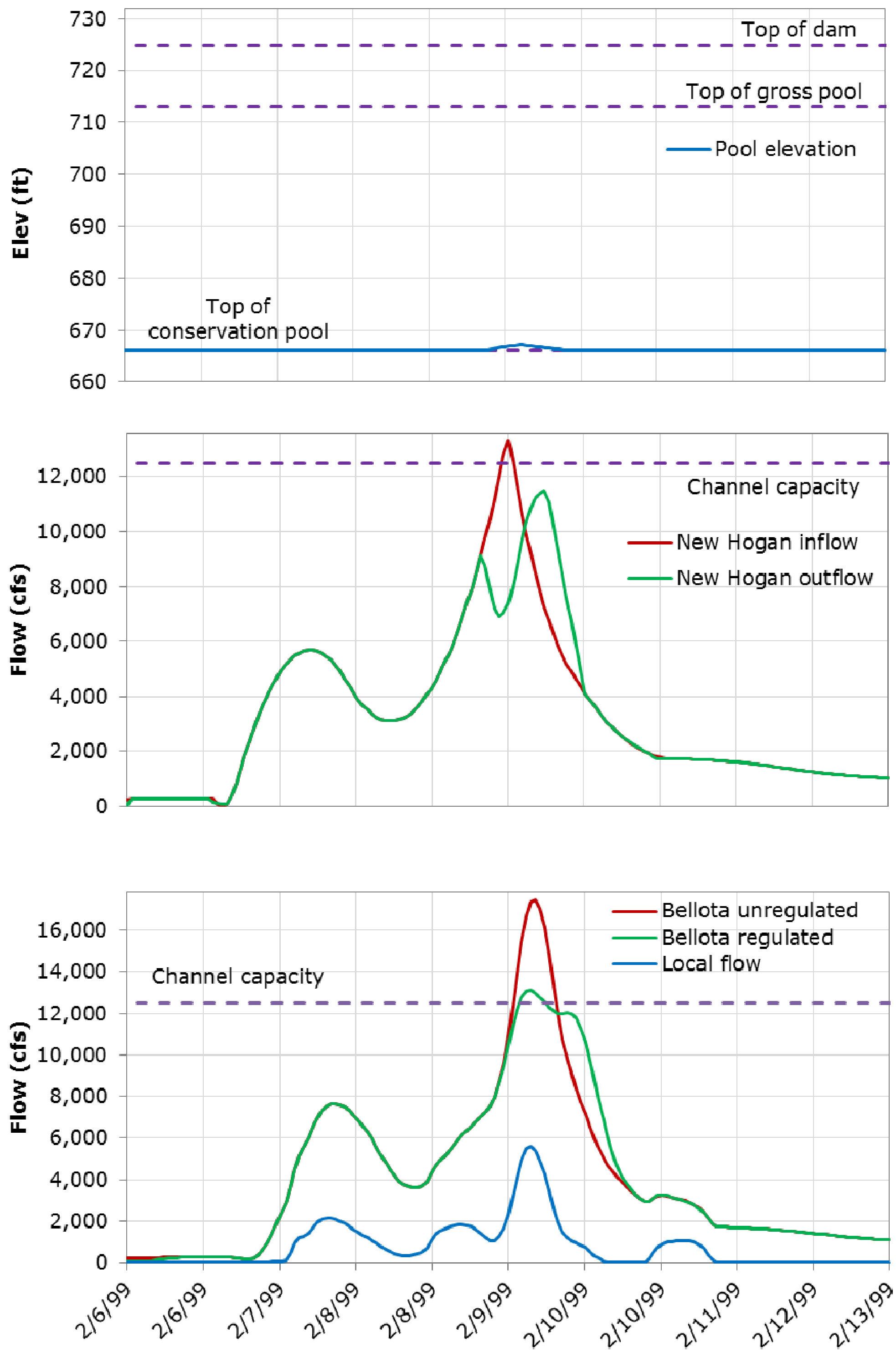


Figure 8. New Hogan Reservoir routing of 1999 event scaled by 1.0

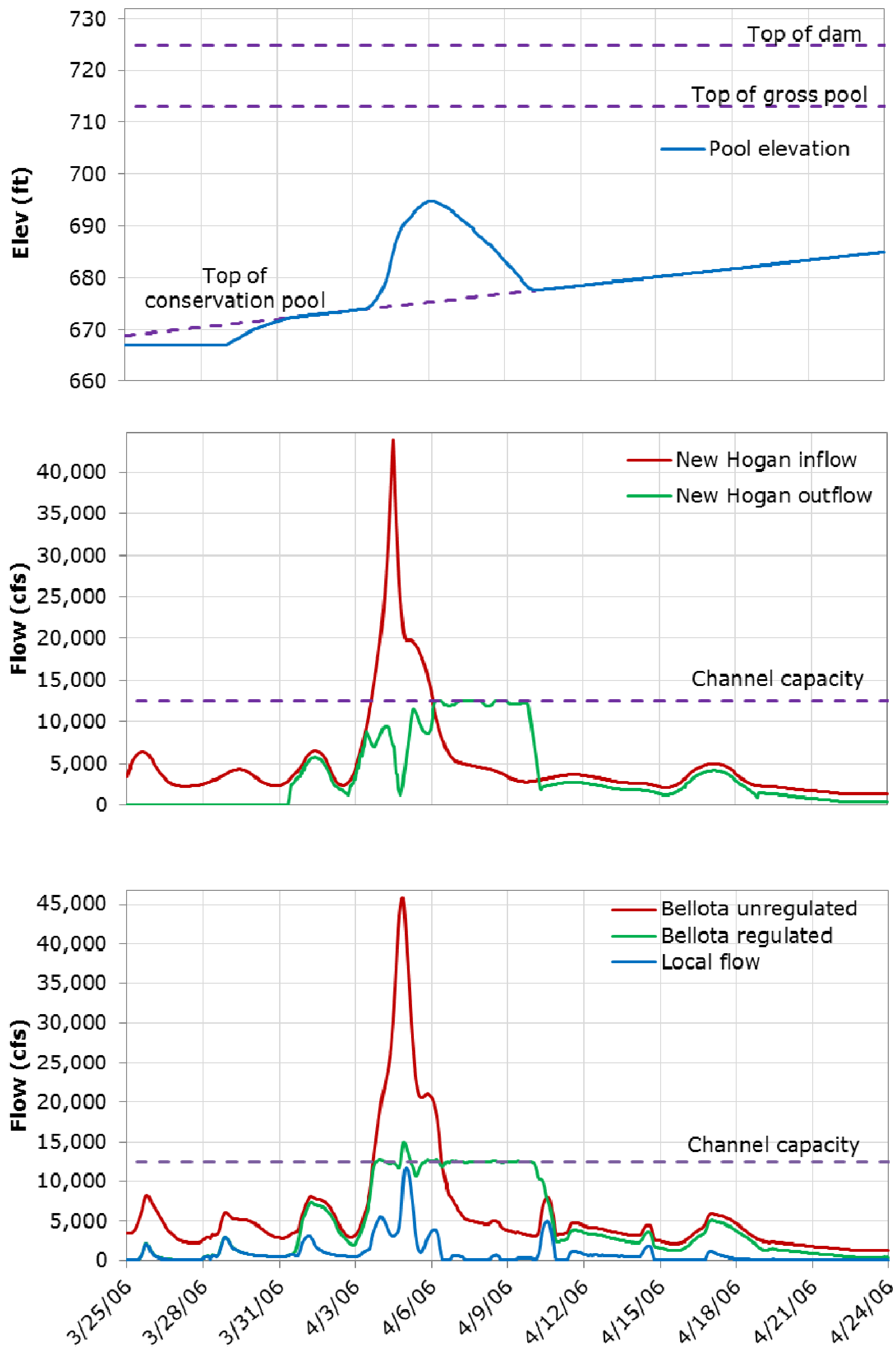


Figure 9. New Hogan Reservoir routing of 2006 event scaled by 1.6

Attachment B. Assessment of coincident reservoir inflow and local flow annual exceedence probabilities

Overview

The unregulated flow-frequency curve is used to predict the flow and volume for the rare events. To transform the unregulated flow and volume to regulated peak flow, we must rely on reservoir simulations of historical or design (scaled) events.

A challenge in this transformation in a regulated system is the distribution of volume above and below the regulating features, in this case a flood control reservoir. Given the variability of historical flood events, typically a predictable or fixed relationship of volume above and below the reservoir does not exist. Thus, this variability must be accounted for in development of the transform.

In the baseline analysis, as documented in the June 2011 report, we followed guidance from EM 1110-2-1415 (USACE 1993), page 3-26:

> *<u>(3) Use of hypothetical-flood routings</u>. Usually recorded values of flows are not large enough to define the upper end of the regulated frequency curve. In such cases, it is usually possible to use one or more large hypothetical floods (whose frequency can be estimated from the frequency curve of unregulated flows) to establish the corresponding magnitude of regulated flows. These floods can be multiples of the largest observed floods or of floods computed from rainfall; but it is best not to multiple any one flood by a factor greater than two or three. The floods are best selected or adjusted to represent about equal severity in terms of runoff frequency of peak and volumes for various durations. The routings should be made under reasonably conservative assumptions as to initial reservoir stages.*

Also of note in the EM regarding local flows is the following:

> *<u>(5) Runoff from unregulated areas</u>. In estimating the frequency of runoff at a location that is a considerable distance downstream from one or more reservoir projects, it must be recognized that none of the runoff from the intermediate areas between the reservoir(s) and the damage center will be regulated. This factor can be accounted for by constructing a frequency curve of the runoff from the intermediate area, and using this curve as an indicator of the lower limit for the curve of regulated flows. Streamflow routing and combing of both the flows from the unregulated area and those from the regulated area is the best procedure for deriving the regulated frequency curve.*

Here, we evaluate the coincidence of events, as related to local flows and reservoir inflows, of the same or similar probabilities for the historical and scaled historical events used in the baseline analysis. To do so, we first construct a local flow-frequency curve and then pair the probability of local flows and reservoir inflows from historical events. Then, we use these figures to assess the relationship with respect to annual exceedence probabilities of local flows and reservoir inflows. This comparison is made for both historical events and scaled historical events that are used in the unregulated to regulated flow transform development.

Local flow-frequency curve development

The peak local flow-frequency curve developed and presented here is for the purpose of assessing the coincidence, with respect to annual exceedence probabilities, of local flows and reservoir inflows. We developed these curves as a comparison tool only and is referred to as the "limited-use local flow-frequency curve." Currently, rainfall-runoff analyses with design precipitation events are being completed to support the development and adoption of a local flow-frequency curve on the Calaveras River for use in the LSJR FS.

The local flow area we are referring to here is the area downstream of New Hogan Reservoir and upstream of Bellota along the Calaveras River. The area is approximately 110 sq. mile and is illustrated in Figure 8 from the June 2011 report.

To develop the peak local flow flow-frequency curve, we:

1. Identified the local peak flow annual maximum series. For this, we used only the peak flows directly computed from observed data. For the peak series of annual maximums, this includes those events where local flows were developed using Option 1 (as defined in section "Unregulated flow time series" of the June 2011 report) and hourly flows (as defined in Table 5 of the June 2011 report). Thus, this includes 14 annual peaks from the 1996 through 2009 water years. The annual maximum series is listed in Table 4.

   [Peak local flows for the other historical events are from the data series smoothing as described in section "Regulated flow time series development," subsection "Smooth unregulated flow time series" of the June 2011 report. These synthetic peaks are not used here for frequency analysis.]

2. Consistent with Corps policy and the standard of practice, fit a Pearson III (LPIII) distribution to the logarithmic transforms of annual maximum series identified from directly calculated local flow data following procedures specified in *Bulletin 17B* (IACWD 1982). We fit the curve using the expected moment algorithm (EMA) enabled flow-frequency software PeakfqSA, version 0.937. This was developed by Tim Cohn of the USGS and is based on the USGS's flow-frequency software PeakFQ (Cohn 2007).For this statistical analysis, we developed and used regional skew values using the relationships developed by the USGS (USGS 2011).

The resulting curve is shown in Figure 10 and selected flow quantiles are tabulated in Table 5.

For this analysis, we used only the directly calculated local flows because those are the values appropriate for peak flow-frequency analysis. Use of the local flows values estimated using Option 2 or Option 3, as described in the June 2011 report, are not appropriate here because:

1. These flows were calculated on a daily basis (as detailed in Table 5 of the June 2011 report) and do not have observations of peak flows.
2. These flows are based on regression analysis where the values are scaled by a factor of approximately 4 or 5 (where the values are estimated as a

function of Cosgrove Creek or reservoir inflow.) The regression analyses used in the June 2011 report were in the context of estimating this local flow contribution to the total watershed runoff volume. Thus, the portion of flow estimated was small in comparison to the total.

As a check of the limited-use local flow-frequency curve, we:

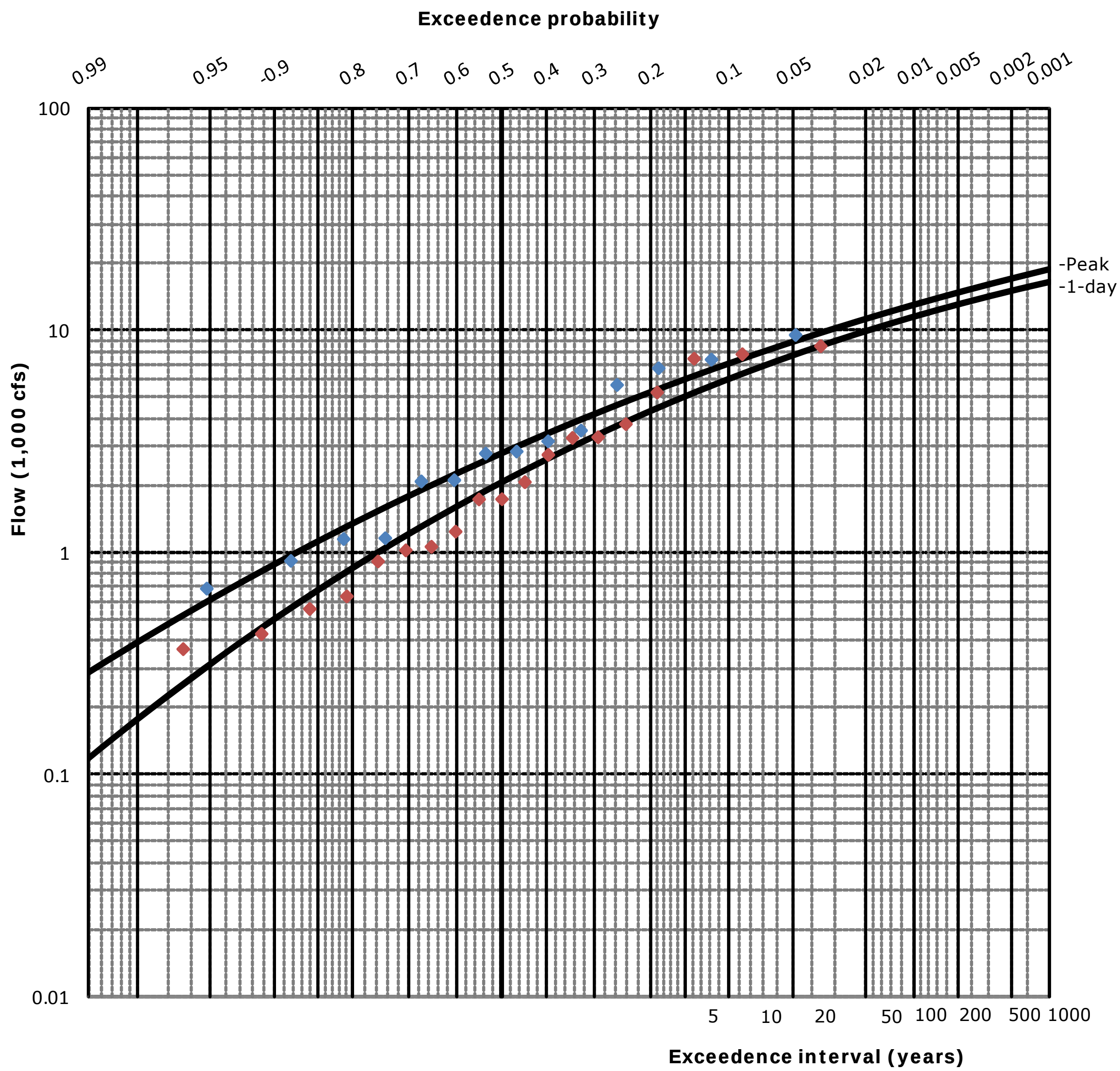
- Fit an unregulated volume-frequency curve to the 1-day local flow volumes following guidance provided by EM 1110-2-1415 (USACE 1993). Here, we used values calculated using Option 1 including those calculated on a daily basis (as defined in Table 5 of the June 2011 report), for a total of 19 annual maximums. This annual maximum series is listed in Table 4. For this statistical analysis, consistent with Corps policy and the standard of practice, fit a Pearson III (LPIII) distribution to the logarithmic transforms of annual maximum series again using PeakfqSA, version 0.937. We used a 1-day regional skew developed using the USGS relationships (USGS forthcoming). In addition, we treated the 19 1-day volumes as a systematic record, assuming no gaps or missing values. The resulting curve is shown in Figure 10 and selected flow quantiles are tabulated in Table 5. We compared the shape of the volume-frequency curve to the peak flow-frequency curve.
- Compared the results to the Cosgrove Creek peak flow-frequency curve from the recent Cosgrove Creek hydrology study (USACE 2010) multiplied by 3.2. This is the factor that is used to estimate the local flow as a function of Cosgrove Creek, Option 2 as described in the June 2011 report. We found the curves to be similar, but the scaled version of the Cosgrove Creek curve to be slightly higher. For the p=0.01 flow the scaled curve was 6% higher and for the p=0.005 flow the scaled curve was 4% higher than the peak curve developed here. (For reference, the Corps' Cosgrove Creek peak flow-frequency curve has the following properties: mean is 2.974, standard deviation is 0.3519, and adopted skew is -0.6.)
- Compared the peak flow and 1-day volume frequency curves on regional flow-per-square-mile estimates. For this, we used frequency curves from the Comp Study (USACE 2001) and latest study on Cosgrove Creek (USACE 2010) for all locations other than those on the Calaveras River and Littlejohn Creek. For these latter locations, we used the results from our June 2011 analyses. Specifically, we: (1) divided the peak, 1-day, and 3-day flow quantiles of 8 nearby watersheds by their associated watershed area, and (2) plotted these values as a function of watershed area. We found the quantiles associated with the local flow to be consistent with these other watersheds. This comparison is illustrated in Figure 11 for the p=0.01 flows and Figure 12 for the p=0.005 flows.
- Compared the peak flow-frequency curve to the results of the baseline analysis. The local flow peak flow-frequency curve should be less than the total flow regulated peak flow-frequency curve at Bellota, consistent with EM 1110-2-1415 guidance, and it is.

Table 4. Annual maximum local flow series used for frequency analysis

Water year (1)	Peak local flow for area along Calaveras River between New Hogan Reservoir and Bellota, CA (cfs) (2)	1-day local flow for area along Calaveras River between New Hogan Reservoir and Bellota, CA (cfs) (3)
1988	—	8,507
1989	—	—
1990	—	1,027
1991	—	7,823
1992	—	3,797
1993	—	7,471
1994	—	—
1995	—	—
1996	2,764	915
1997	6,688	3,312
1998	9,436	5,267
1999	5,620	2,762
2000	3,136	1,740
2001	2,069	1,066
2002	2,096	1,246
2003	681	432
2004	2,819	1,744
2005	3,505	2,081
2006	7,312	3,290
2007	1,149	369
2008	1,138	560
2009	908	638

Table 5. Calaveras River limited-use frequency curves: local flow between New Hogan Reservoir and Bellota, CA

Annual exceedence probability (1)	1/annual exceedence probability (2)	Peak flow (cfs) (3)	1-day volume (cfs) (4)
0.500	2	2,817	2,067
0.200	5	5,310	4,324
0.100	10	7,134	6,015
0.050	20	8,942	7,688
0.020	50	11,318	9,855
0.010	100	13,103	11,449
0.005	200	14,874	12,995
0.002	500	17,188	14,957



Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.421	0.355	-0.492
1-day	3.270	0.427	-0.644

Notes:

- Median plotting positions.
- Drainage area: 110 sq. miles.
- Record lengths:
  Peak flows: 14 years.
  1-day volumes: 19 years.
- Regional skew values developed by USGS.

Figure 10. Limited-use peak local flow-frequency curve for areas along Calaveras River from New Hogan Reservoir to Bellota, CA

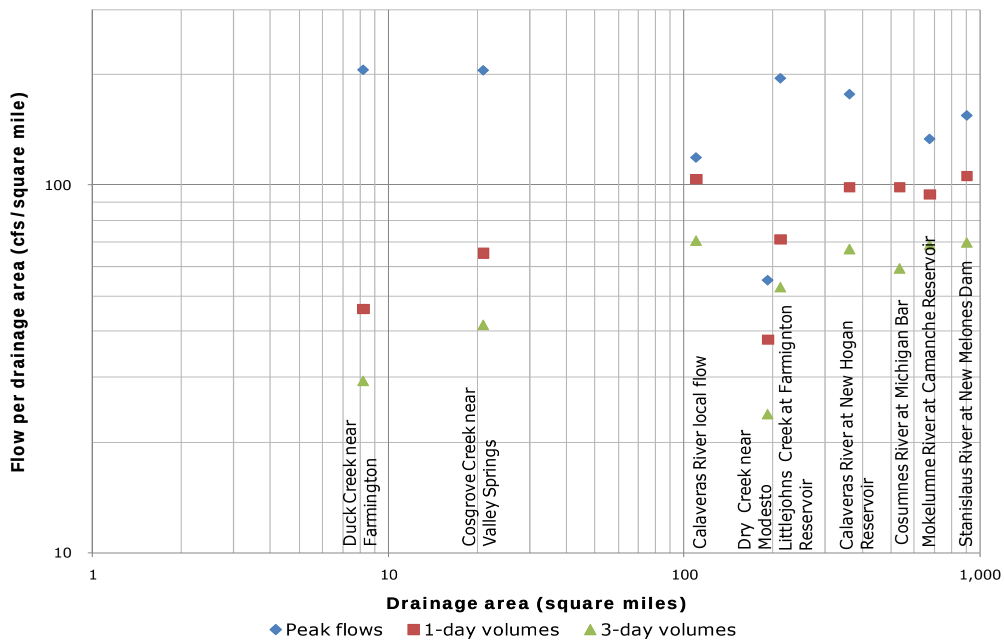


Figure 11. Comparison of regional flow per square mile ratios for the p=0.01 event; values from June 2011 report for Calaveras River and Littlejohn Creek, Comp Study (USACE 2001), and Cosgrove Creek study (USACE 2010)

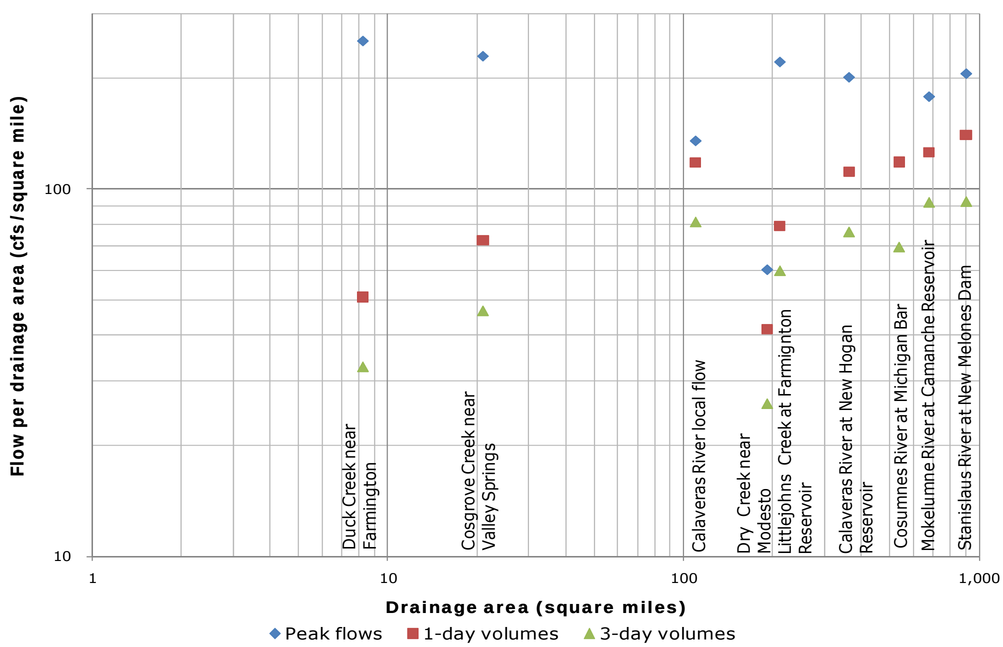


Figure 12. Comparison of regional flow per square mile ratios for the p=0.005 event; values from June 2011 report for Calaveras River and Littlejohn Creek, Comp Study (USACE 2001), and Cosgrove Creek study (USACE 2010)

Coincident event probabilities between local flow and reservoir inflow

Using the local flow-frequency curve in Figure 10 and the New Hogan reservoir inflow-frequency curve from the June 2011 report, we calculated and compared the annual exceedence probability (AEP) of the local flow and the coincident unregulated reservoir inflow for the selected events. We completed these comparisons considering the following combinations of flows, volumes, and events:

- Peak local flow and peak reservoir inflow for all 104 historical events. This is the entire period of record used for the Calaveras River frequency curves shown in the June 2011 report and is shown in Figure 13. As described in the June 2011 report, various computational options were used to estimate the local flow series for the historical events based on the availability of gage data. In the figure, we note by historical event which computational option was used for estimating that event's local flow.
- Peak local flow and 3-day reservoir inflow for 104 historical events. This is shown in Figure 14 and is similar to the comparison above, but here the 3-day reservoir value is used rather than the peak inflow. The values in the figure compare similarly between Figure 13 and Figure 14.
- Peak local flow and peak reservoir inflow for 8 scaled events. Here we focus on a combination of historical and scaled historical events, the same events as those listed in Table 2. This is shown in Figure 15.
- Peak local flow and 3-day reservoir inflow for 8 scaled events. This is shown in Figure 16 and is similar to the comparison above, but here the 3-day reservoir value is used rather than the peak inflow.
- Peak local flow and peak reservoir inflow for the 190 scaled historical events used to develop the flow transforms detailed in the baseline analysis. This is shown in Figure 17. Again, as described in the June 2011 report, various computational options were used to estimate the local flow series for the historical events based on the availability of gage data. In the figure, we note by historical event (which affects the scaled version of the historical event) which computational option was used for estimate the event's local flow.
- Peak local flow and 3-day reservoir inflow for the 190 scaled historical events used to develop the flow transforms detailed in the baseline analysis. This is shown in Figure 18 and is similar to the comparison above, but here the 3-day reservoir value is used rather than the peak inflow.

This analysis of coincidence AEP flows illustrates that there is not a consistent relation between the local flow and the reservoir inflow AEP values. In these figures, the area below the gray dashed "1 to 1" line indicates a region where the local flow AEP is greater than the reservoir inflow AEP. The area above the line indicates a region where the local flow AEP is less than the reservoir inflow AEP. On average for both the historical events and the scaled historical events, the local flow AEP tends to be greater than the reservoir inflow AEP. For example, when the reservoir inflow is a p=0.01 (100-yr) flow, the coincident local flow may be a p=0.10 (10-yr) flow. This is best illustrated in Figure 14 and Figure 18.

In Figure 17 and Figure 18, the events plotted appear to exhibit several linear and curved trends. The trends are scaled versions of a given historical event. Recall that historical events are scaled uniformly by specified factors, as described in the June 2011 report. The trends seen in the figures are a function of the relationship between the frequency curves used to calculate the probabilities of the peak local flows and the coincident event reservoir inflows. In Figure 19 and Figure 20, we highlight these trends with a select number of the historical events that were scaled and simulated to develop the unregulated to regulated flow transforms.

[For the baseline analysis, the local flow and reservoir inflow series were derived based on daily values. A data smoothing algorithm was then used to create synthetically an hourly series. The exception to this is where hourly data were available to derive the hourly local flow series directly. In this case, the derived hourly series was used directly. This flow development process is described in the June 2011 report. Here, the peak values from the baseline series are used.]

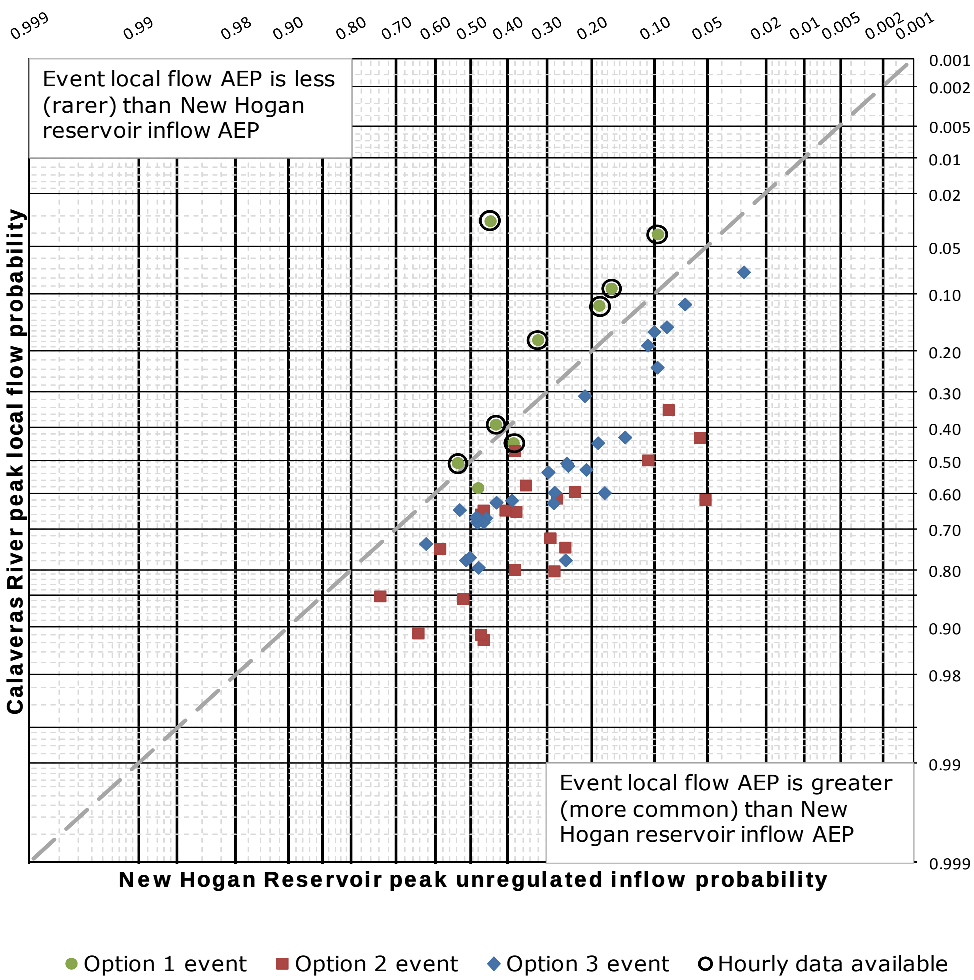


Figure 13. New Hogan Reservoir peak inflow and Calaveras River peak local flow coincident event probabilities: historical events

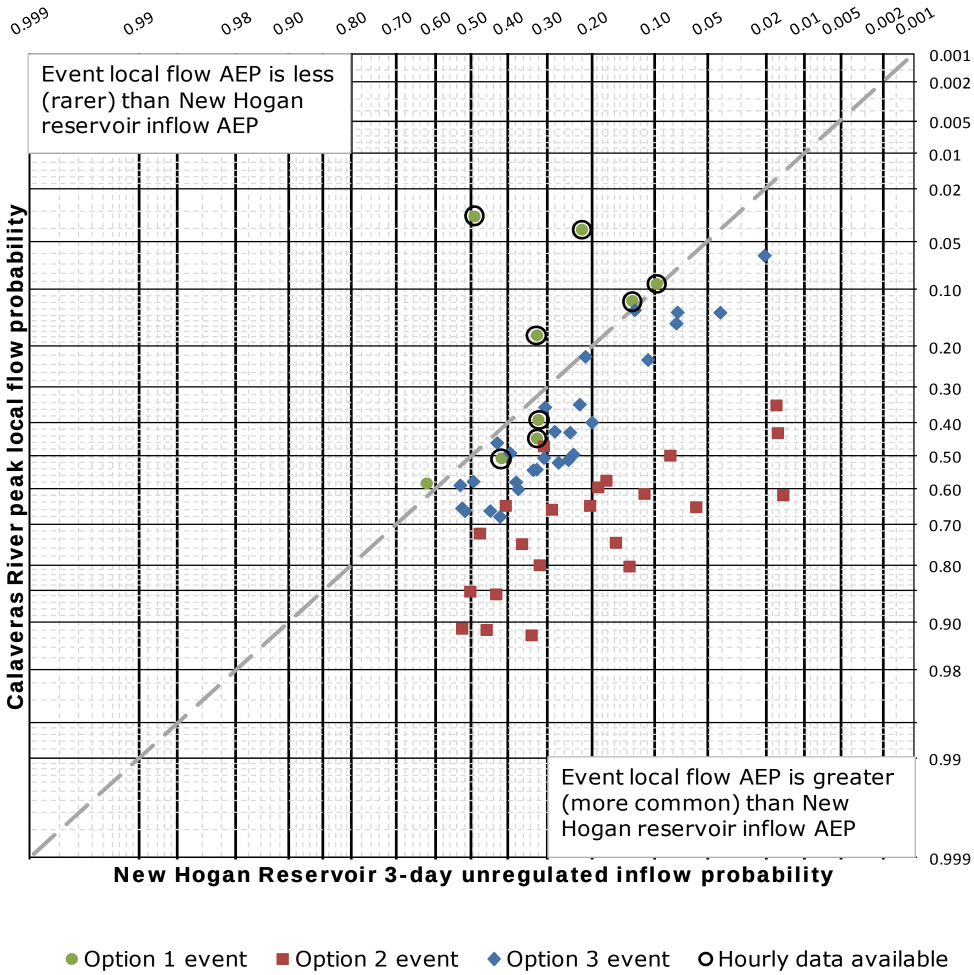


Figure 14. New Hogan Reservoir 3-day inflow volume and Calaveras River peak local flow coincident event probabilities: historical events

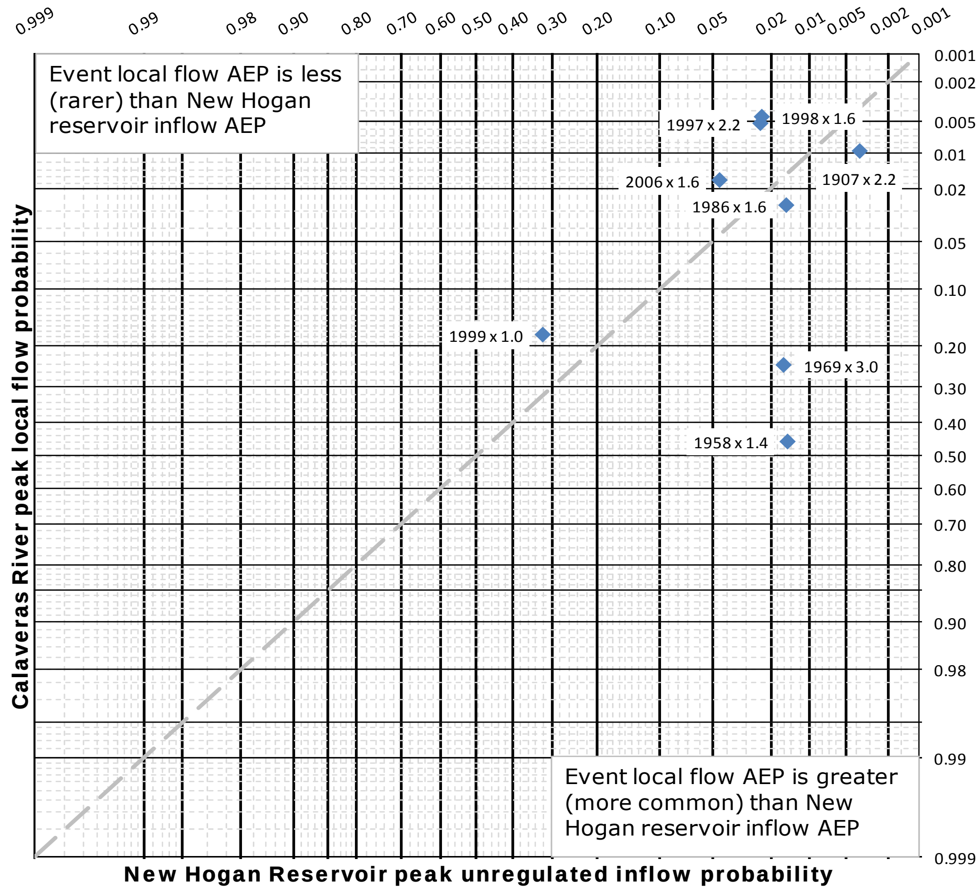


Figure 15. New Hogan Reservoir peak inflow and Calaveras River peak local flow coincident event probabilities: selected scaled events

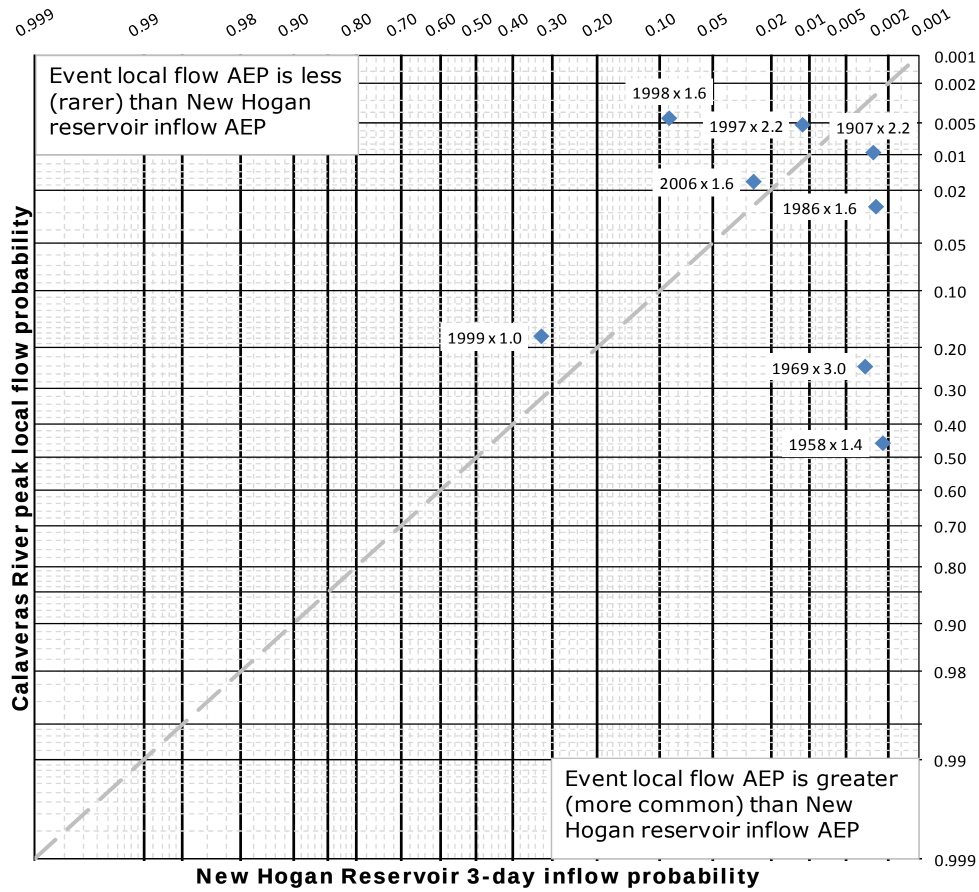


Figure 16. New Hogan Reservoir 3-day inflow volume and Calaveras River peak local flow coincident event probabilities: selected scaled events

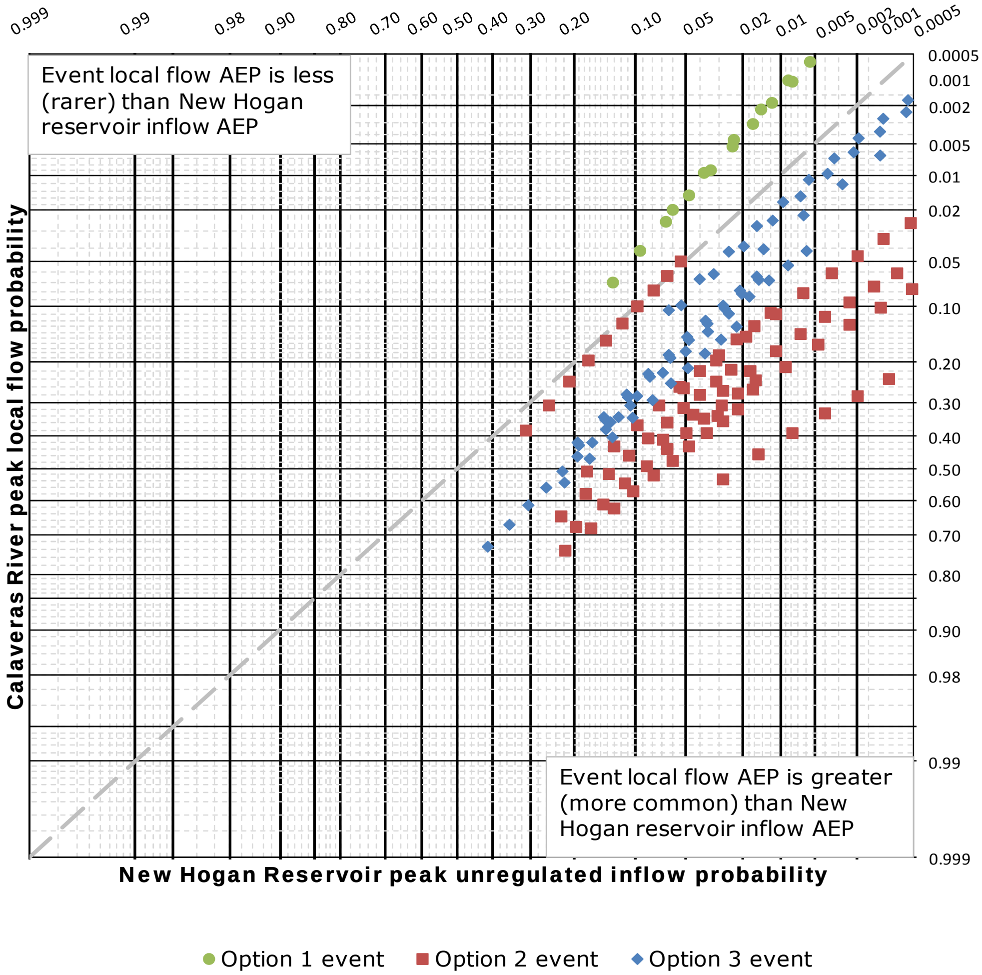


Figure 17. New Hogan Reservoir peak inflow and Calaveras River peak local flow coincident event probabilities: scaled events used to develop baseline flow transform

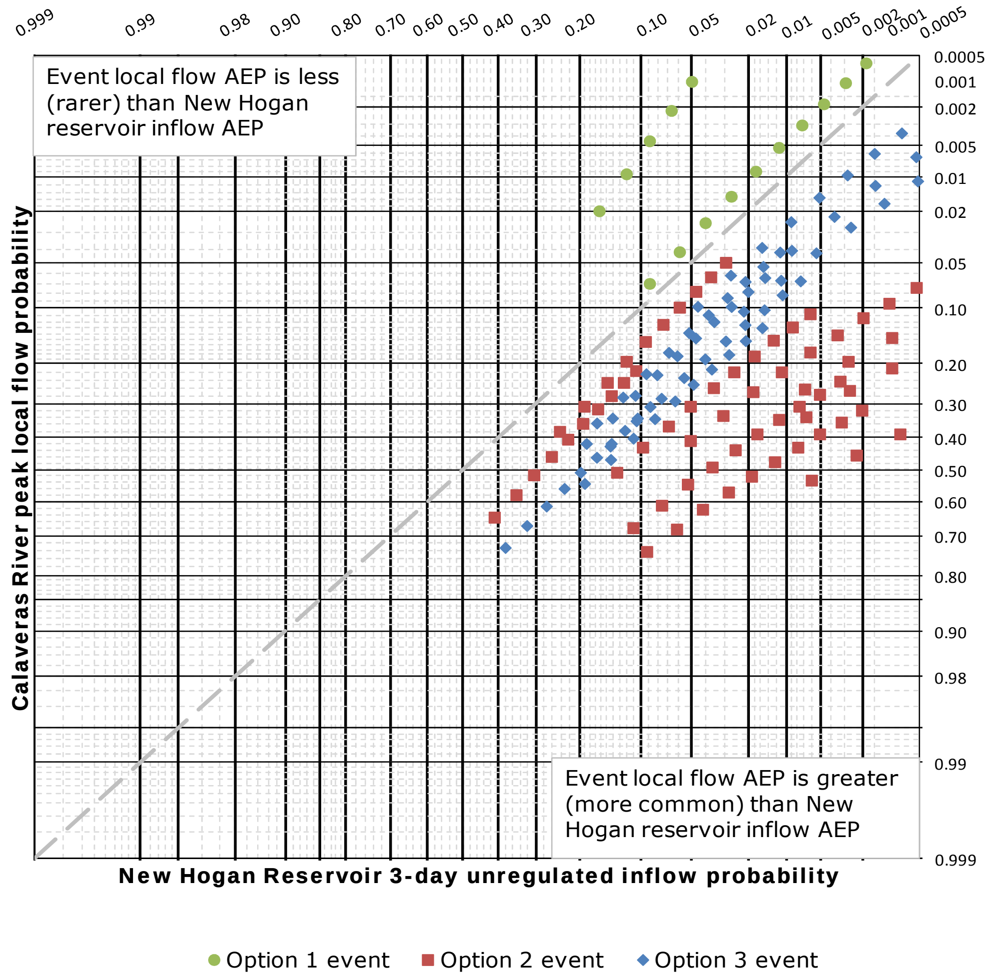


Figure 18. New Hogan Reservoir 3-day inflow volume and Calaveras River peak local flow coincident event probabilities: scaled events used to develop baseline flow transform

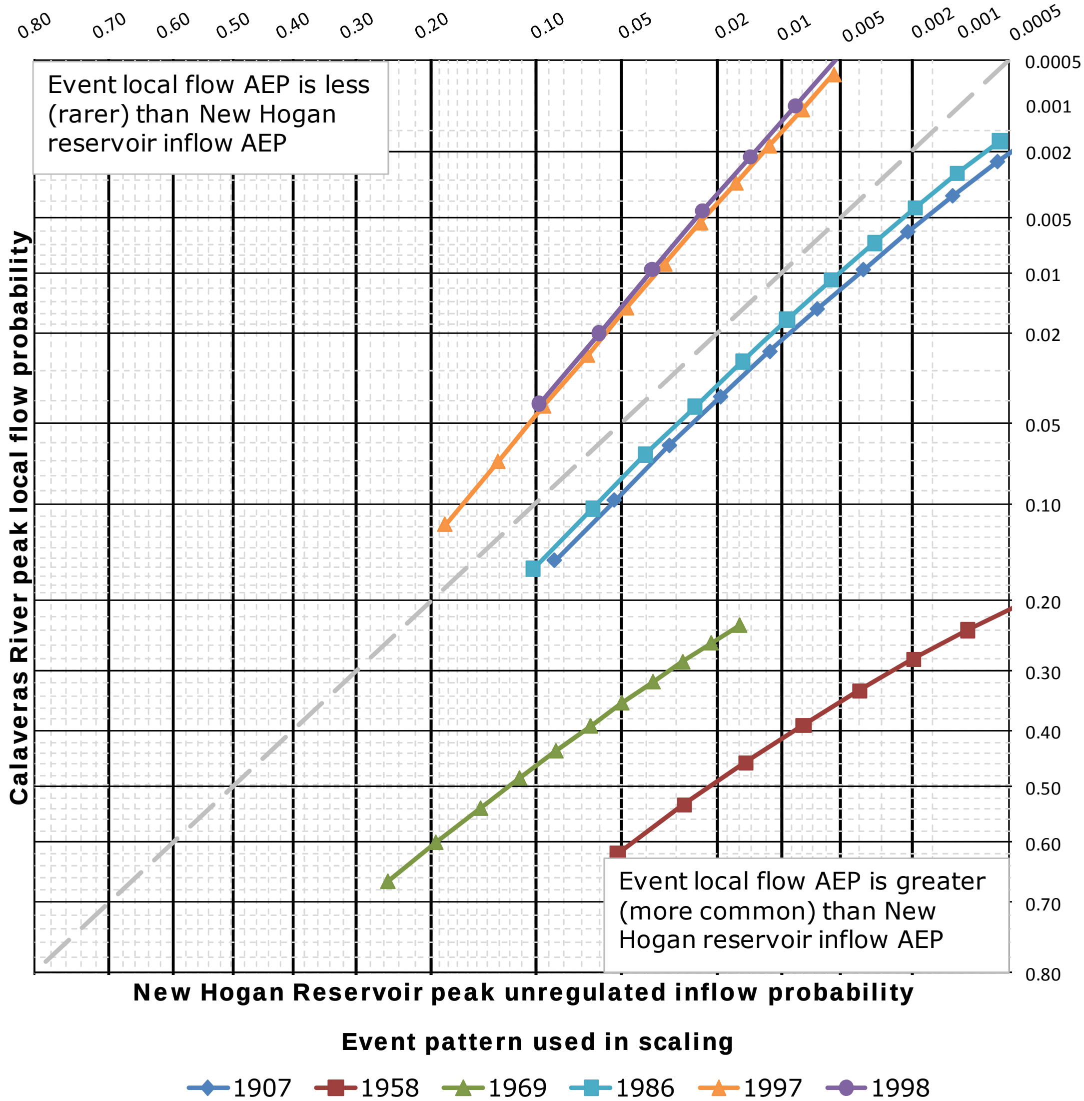


Figure 19. New Hogan Reservoir peak inflow and Calaveras River peak local flow coincident event probabilities: traces of select scaled events used to develop baseline flow transform

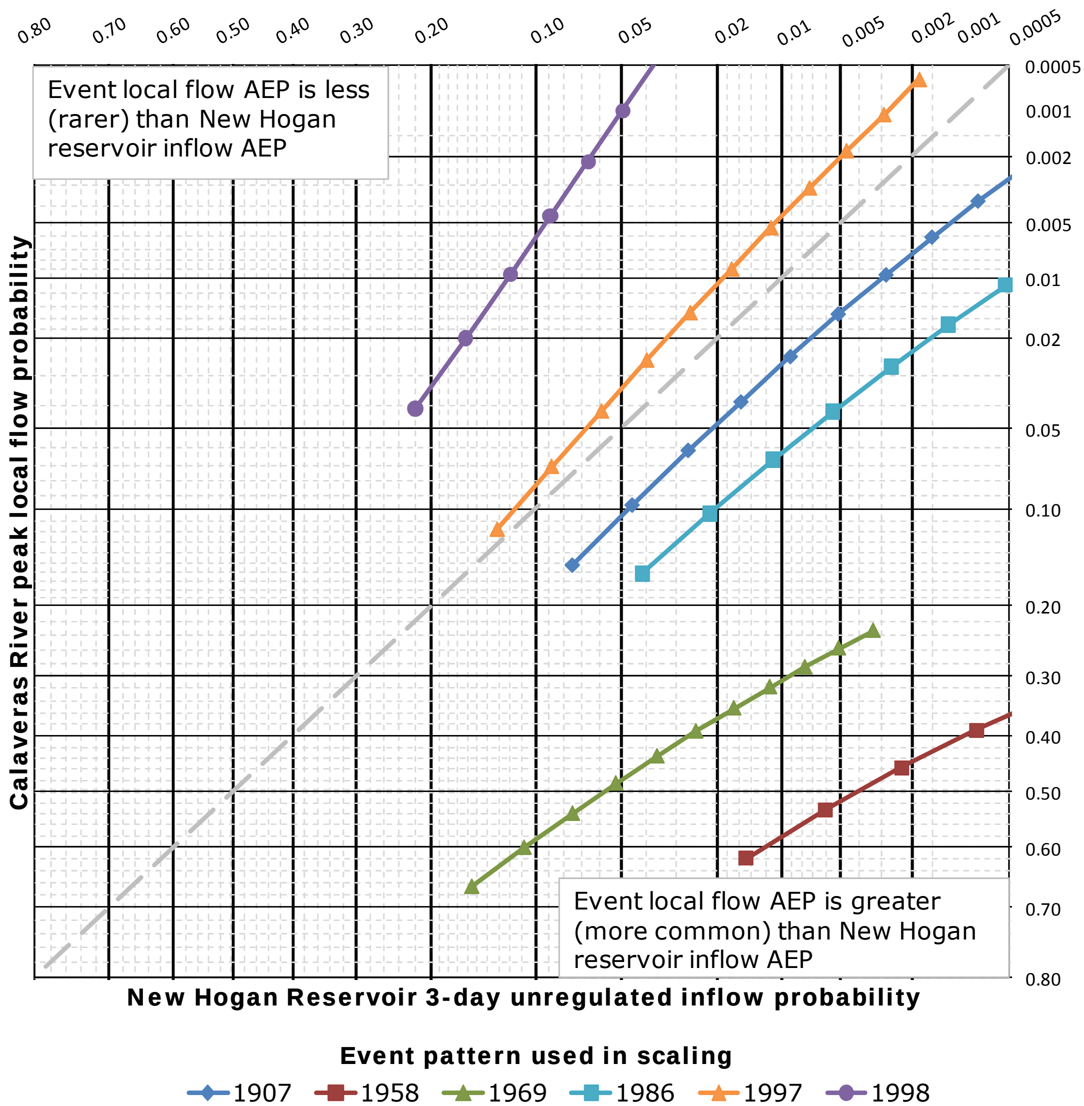


Figure 20. New Hogan Reservoir 3-day inflow volume and Calaveras River peak local flow coincident event probabilities: traces of select scaled events used to develop baseline flow transform

Attachment C. Reservoir simulation of design (scaled) events

Overview

To evaluate the sensitivity of reservoir flood storage and the effect of the uncontrolled local flows to the peak regulated flow-frequency curve at Bellota, we developed and evaluated an array of design (scaled) events. These design events (or hydrographs) are historical events scaled to a specific peak and/or volume(s) of specified probability. Here, we developed design events focused at p=0.005 flow at New Hogan and Bellota using consistent methodology as the baseline analysis described in the June 2011 report. We also developed and simulated design (scaled) events for the p=0.01 and p=0.002 flows at both locations.

Design hydrograph development and reservoir simulation

We developed design hydrographs at both New Hogan Reservoir and at Bellota. Thus, the design hydrographs for New Hogan Reservoir use the New Hogan unregulated flow-duration-frequency curve and the design hydrographs for Bellota use the Bellota unregulated flow-duration-frequency curve as documented in the June 2011 report.

To develop the design hydrographs for New Hogan Reservoir, we:

1. Selected historical events to serve as the template of the design hydrograph. Each historical event contains information including the temporal distribution (hydrograph shape and timing) and the spatial distribution (balance of flow above and below the reservoir). Here we used the 1958, 1986, 1997, 1998, and 2006 events.
2. Specified an AEP. We started with an AEP of 0.01.
3. Evaluated the AEPs of flow-duration properties of the selected historical hydrographs, using the New Hogan Reservoir flow-duration-frequency curve from the June 2011 report.
4. Scaled each selected event to a specified flow duration value. We started with duration equal to 3 days. For the scaling, both the reservoir inflow and the local flow are scaled uniformly. The uniform scale factor is computed as the specific flow-duration value from the unregulated flow-frequency curve matching the selected AEP divided by the corresponding flow-duration value from the historical event.
5. Simulated operation of the event with HEC-ResSim.
6. Recorded peak releases and downstream flows for each simulation.
7. Selected a different design duration and repeated steps 4 through 6. We repeated this process for durations of 4, 5, and 6 days in addition to the duration of 3 days.
8. Selected another AEP and repeated steps 3 though 7. We repeated this process for AEPs of 0.005 and 0.002 in addition to 0.01.

We repeated the process above for an analysis focused on flows at Bellota. Thus, the same steps were used but the Bellota unregulated flow-frequency curve from the June 2011 report was used instead of the New Hogan Reservoir inflow curve.

Following the process above, we developed and simulated 180 events. This includes 6 historical events, 3 design AEPs, 5 design durations, and 2 locations (unregulated flow-frequency curve). The results of these simulations are shown in Table 7 through Table 18.

Reservoir simulation results and synthesis

Below, by annual exceedence probability, we summarize our findings from the design (scaled) event simulations. Selective plots of the simulation of the design events, specifically those using the New Hogan frequency curve, are included. Additional plots of simulations using the Bellota frequency curve are on the CD delivered to the Corps.

For reference, we include Table 6 which describes the routing of the historical event used as a pattern for the design (scaled) events described herein.

Table 6. For reference, routing of historical events (no scaling)

Event (1)	Peak regulated flow at Bellota (cfs) (2)	Peak local flow at Bellota (cfs) (3)	New Hogan peak inflow (cfs) (4)	New Hogan peak release (cfs) (5)
1958	12,533	2,193	50,300	12,457
1986	12,500	5,850	35,500	12,244
1997	13,192	6,688	25,100	12,500
1998	13,422	9,436	25,300	12,500
2006	13,286	7,312	27,400	12,500

Baseline evaluation of p=0.01 design events

Table 7 includes simulation results for all durations for the p=0.01 design events scaled using the New Hogan frequency curve. Table 8 includes simulation results for all durations for the p=0.01 design event scaled using the Bellota frequency curve. Figure 21 through Figure 25 show reservoir routings of the 3-day design duration for 5 patterned events, scaled to p=0.01 flows using the New Hogan frequency curve. Although not included here, plots for all durations using both the New Hogan and Bellota frequency curves are on the CD delivered to the Corps.

The plots show that channel capacity of 12,500 cfs at Bellota is not exceeded for the 1958 and 1986 p=0.01 design events (scaled using either the New Hogan or Bellota frequency curve).

Channel capacity at Bellota is exceeded for all p=0.01 1997, 1998, and 2006 events (scaled using either the New Hogan or Bellota frequency curve). The channel capacity is exceeded because local flows at Bellota are greater than channel capacity.

The p=0.01 design (scaled) events for the 3-day duration are summarized in Table 9 and Table 10.

Table 7. p=0.01 design events scaled using the New Hogan frequency curve

Event pattern (1)	**Duration (days) (2)**	**Scale factor (3)**	**Peak regulated flow at Bellota (cfs) (4)**	**Peak local flow (cfs) (5)**	**New Hogan peak inflow (cfs) (6)**	**New Hogan peak release (cfs) (7)**
1958	3	1.09	12,500[1]	2,390	54,827	12,461
	4	1.06	12,500[1]	2,325	53,318	12,460
	5	1.04	12,500[1]	2,281	52,312	12,459
	6	1.01	12,500[1]	2,215	50,803	12,458
	7	0.99	12,500[1]	2,171	49,797	12,457
1986	3	1.43	12,500[1]	8,366	50,765	12,371
	4	1.32	12,500[1]	7,722	46,860	12,361
	5	1.32	12,500[1]	7,722	46,860	12,361
	6	1.36	12,500[1]	7,956	48,280	12,369
	7	1.41	12,500[1]	8,249	50,055	12,372
1997	3	2.25	13,192	15,073[2]	56,475	12,500[1]
	4	2.34	16,483	15,676[2]	58,734	12,500[1]
	5	2.43	16,961	16,279[2]	60,993	12,500[1]
	6	2.47	17,175	16,547[2]	61,997	12,500[1]
	7	2.53	17,497	16,948[2]	63,503	12,500[1]
1998	3	3.01	28,511	28,402[2]	76,153	14,723
	4	2.87	27,220	27,081[2]	72,611	12,500[1]
	5	2.55	24,280	24,062[2]	64,515	12,500[1]
	6	2.40	22,833	22,646[2]	60,720	12,500[1]
	7	2.39	22,741	22,552[2]	60,467	12,500[1]
2006	3	1.96	15,974	14,332[2]	53,704	12,500[1]
	4	2.05	16,233	14,990[2]	56,170	12,500[1]
	5	2.13	16,616	15,575[2]	58,362	12,500[1]
	6	2.21	18,089	16,160[2]	60,554	12,500[1]
	7	2.24	18,211	16,379[2]	61,376	12,500[1]

Notes:
1. Reservoir release adjusted by hand to 12,500 cfs to compensate for routing problem in HEC-ResSim. There is sufficient storage to contain event.
2. Local flow is greater than 12,500 cfs.

Table 8. p=0.01 design events scaled using the Bellota frequency curve

Event pattern (1)	Duration (days) (2)	Scale factor (3)	Peak regulated flow at Bellota (cfs) (4)	Peak local flow (cfs) (5)	New Hogan peak inflow (cfs) (6)	New Hogan peak release (cfs) (7)
1958	3	1.18	12,500[1]	2,588	59,354	12,466
	4	1.15	12,500[1]	2,522	57,845	12,465
	5	1.12	12,500[1]	2,456	56,336	12,466
	6	1.10	12,500[1]	2,413	55,330	12,466
	7	1.08	12,500[1]	2,369	54,324	12,465
1986	3	1.35	12,500[1]	7,898	47,925	12,368
	4	1.25	12,500[1]	7,313	44,375	12,343
	5	1.24	12,500[1]	7,254	44,020	12,337
	6	1.28	12,500[1]	7,488	45,440	12,351
	7	1.33	12,500[1]	7,781	47,215	12,364
1997	3	2.14	15,339	14,313[2]	53,714	12,500[1]
	4	2.25	16,040	15,049[2]	56,475	12,500[1]
	5	2.32	16,377	15,517[2]	58,232	12,500[1]
	6	2.37	16,643	15,851[2]	59,487	12,500[1]
	7	2.43	16,961	16,253[2]	60,993	12,500[1]
1998	3	2.63	25,014	24,817[2]	66,539	12,500[1]
	4	2.52	24,004	23,779[2]	63,756	12,500[1]
	5	2.29	21,836	21,608[2]	57,937	12,500[1]
	6	2.20	21,033	20,759[2]	55,660	12,500[1]
	7	2.20	21,033	20,759[2]	55,660	12,500[1]
2006	3	1.87	15,539	13,673[2]	51,238	12,500[1]
	4	1.97	16,024	14,404[2]	53,978	12,500[1]
	5	2.07	16,328	15,135[2]	56,718	12,500[1]
	6	2.12	16,568	15,500[2]	58,088	12,500[1]
	7	2.16	16,761	15,793[2]	59,184	12,500[1]

Notes:
1. Reservoir release rounded by hand to 12,500 cfs to compensate for routing problem in HEC-ResSim. There is sufficient storage to contain event.
2. Local flow is greater than 12,500 cfs.

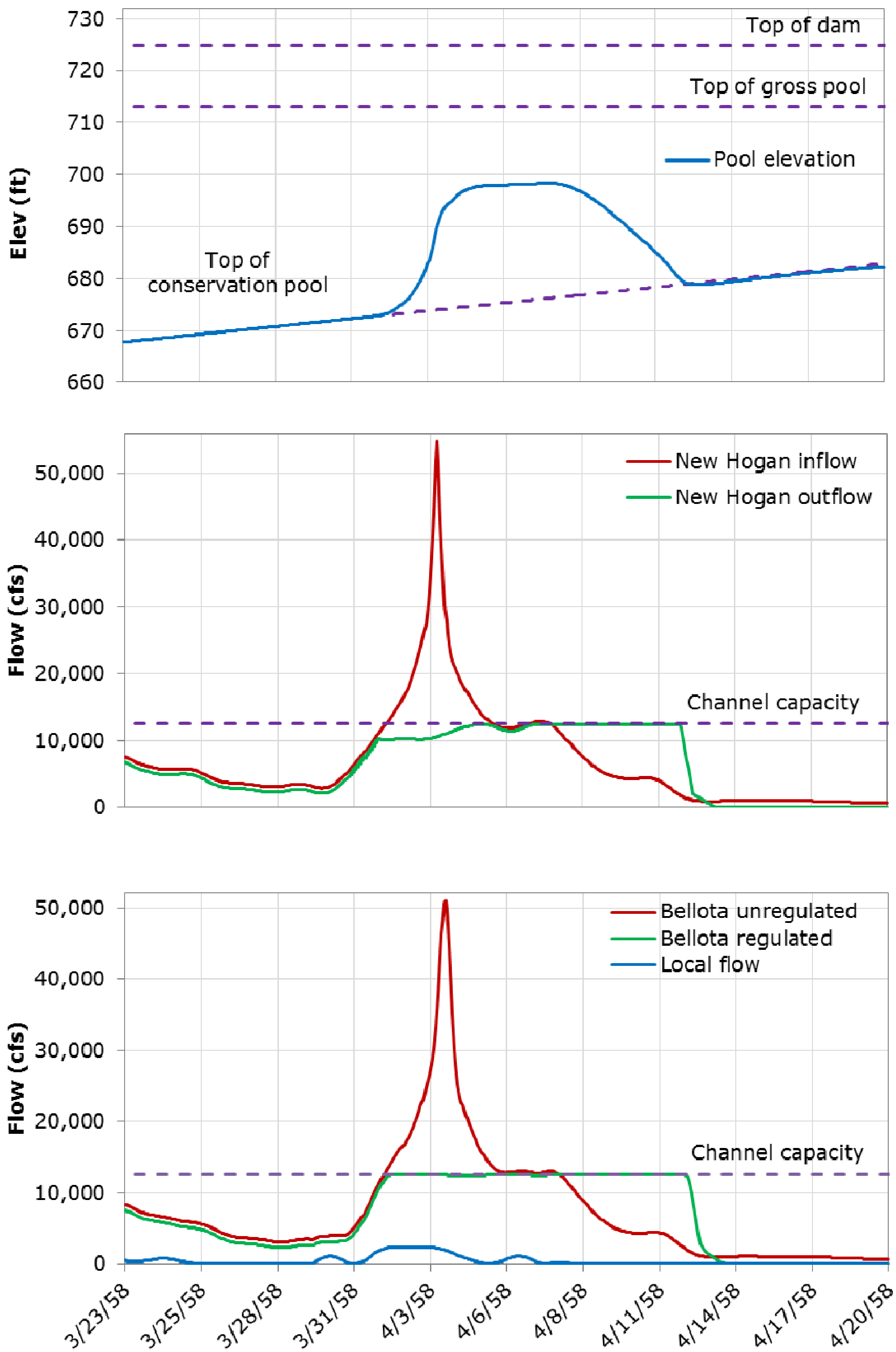


Figure 21. Reservoir routings of the 1958 event scaled using the New Hogan frequency curve to the p=0.01 3-day flow

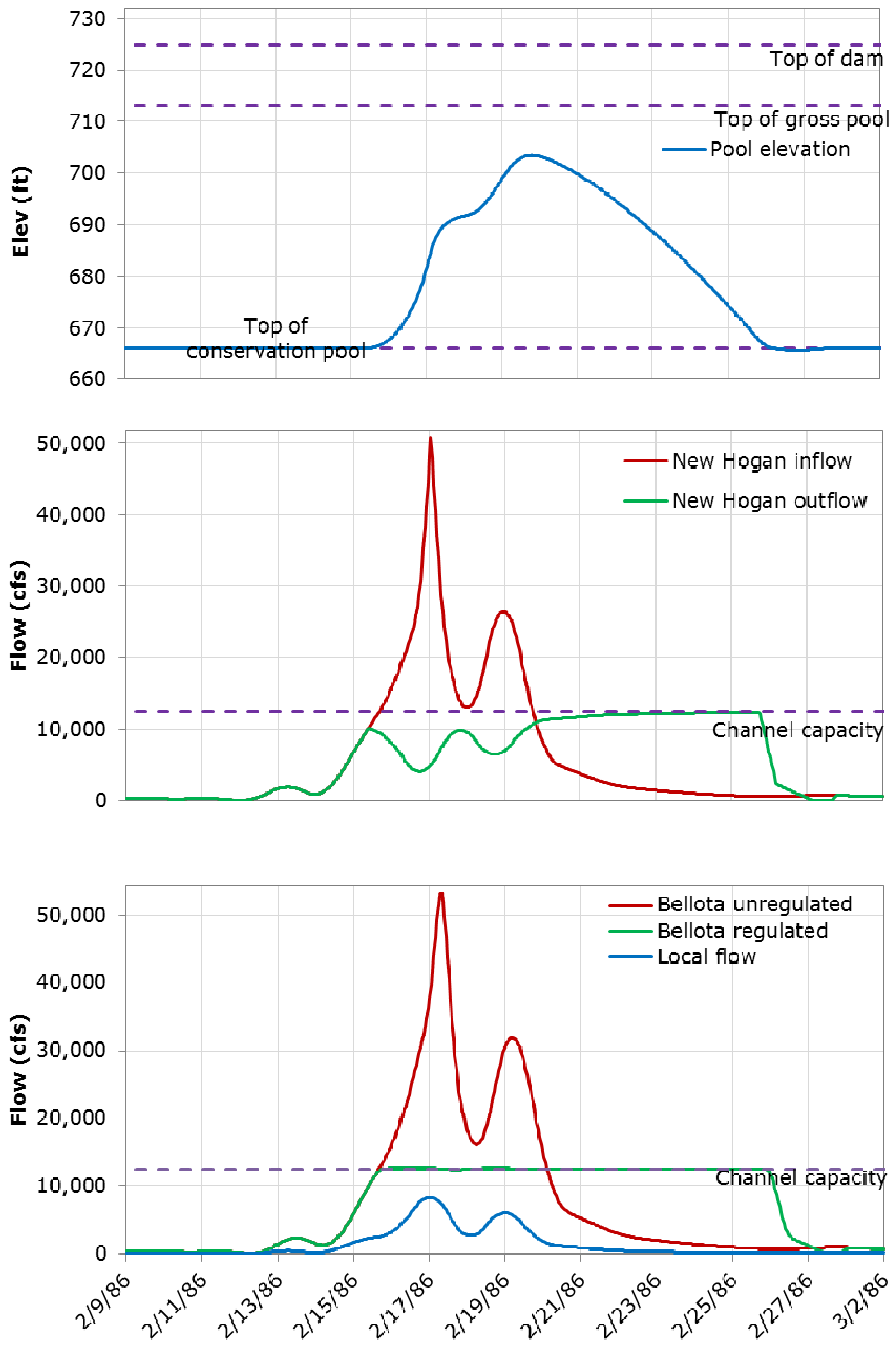


Figure 22. Reservoir routings of the 1986 event scaled using the New Hogan frequency curve to the p=0.01 3-day flow

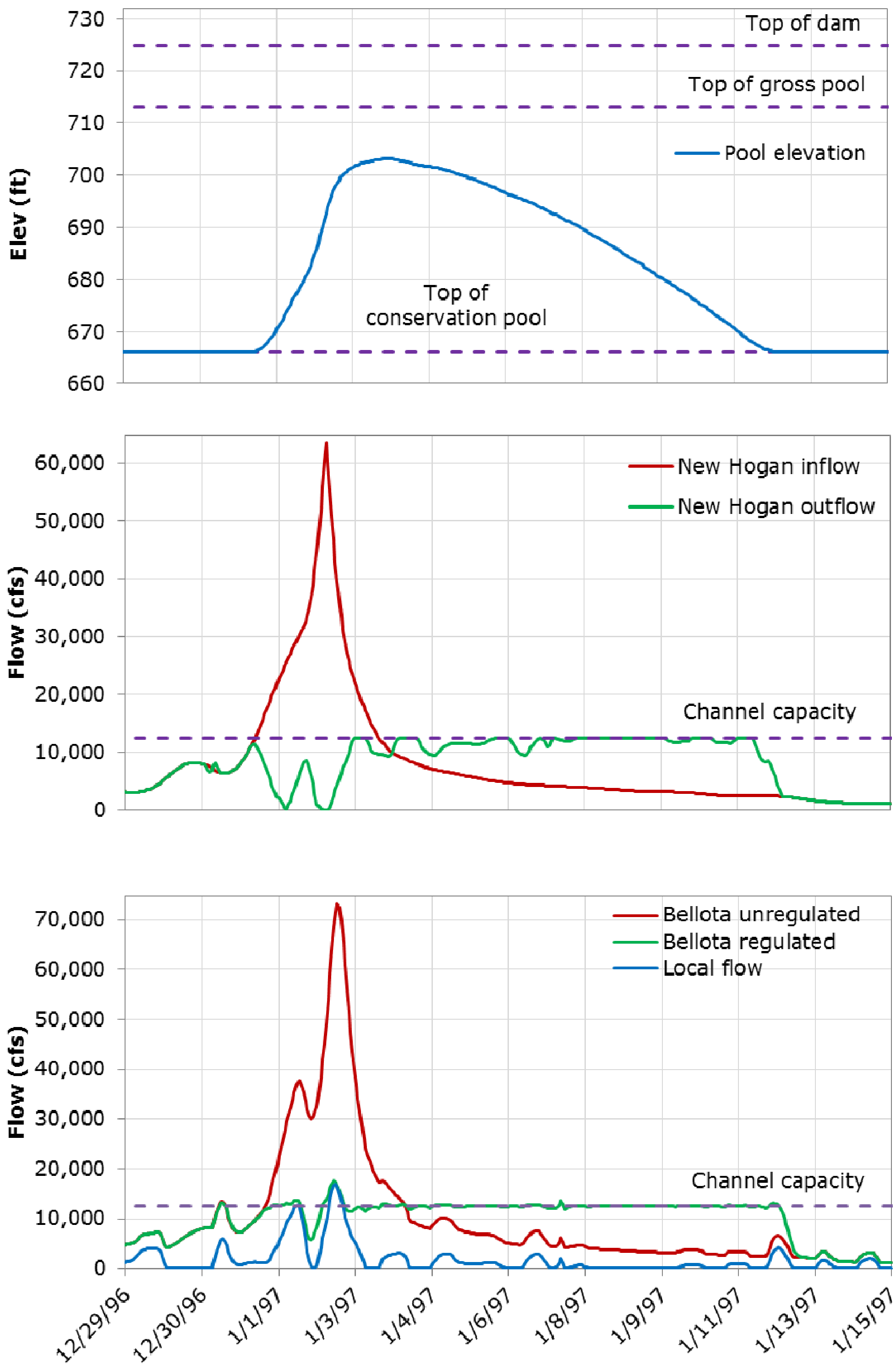


Figure 23. Reservoir routings of the 1997 event scaled using the New Hogan frequency curve to the p=0.01 3-day flow

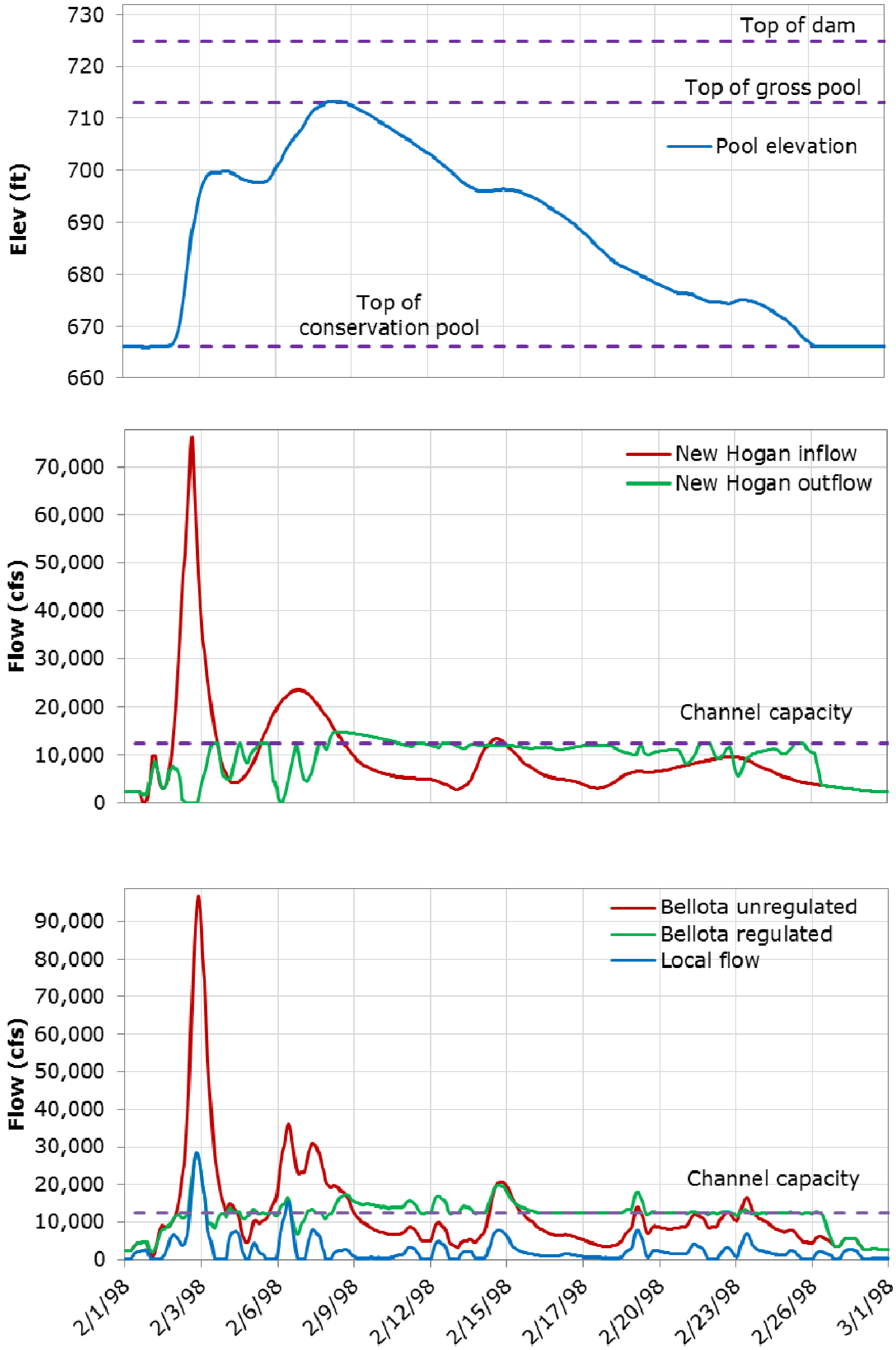


Figure 24. Reservoir routings of the 1998 event scaled using the New Hogan frequency curve to the p=0.01 3-day flow

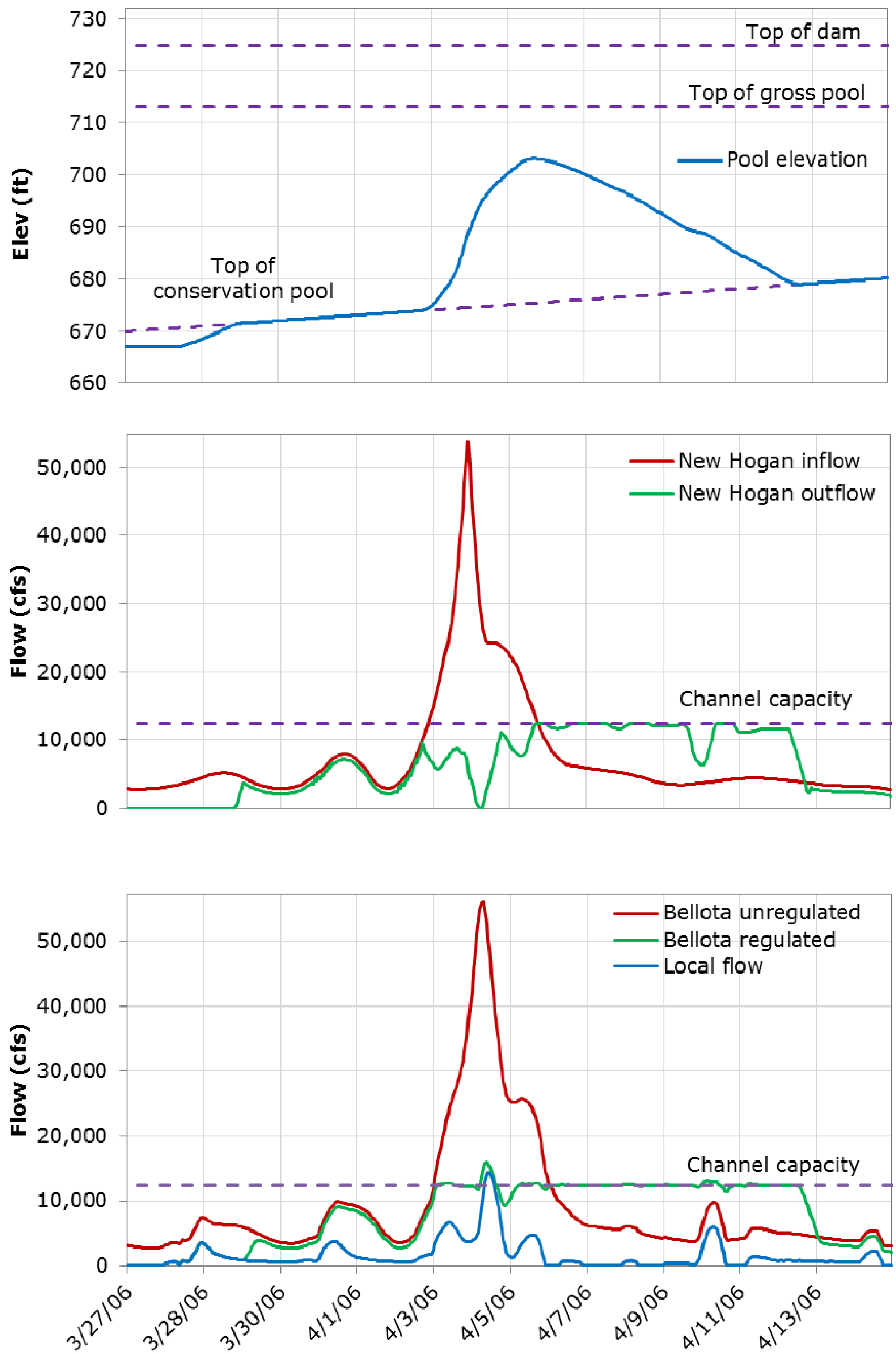


Figure 25. Reservoir routings of the 2006 event scaled using the New Hogan frequency curve to the p=0.01 3-day flow

Table 9. Summary of simulation results for all events analyzed; all flows scaled to the 3-day p=0.01 flow using the New Hogan frequency curve

Pattern event (1)	Channel capacity at Bellota exceeded? (2)	Notes (3)
1958	No	—
1986	No	—
1997	Yes	Peak local flow is greater than channel capacity for all durations.
1998	Yes	Peak local flow is greater than channel capacity for all durations.
2006	Yes	Peak local flow is greater than channel capacity for all durations.

Table 10. Summary of simulation results for all events analyzed; all flows scaled to the 3-day p=0.01 using the Bellota frequency curve

Pattern event (1)	Channel capacity at Bellota exceeded? (2)	Notes (3)
1958	No	—
1986	No	—
1997	Yes	Peak local flow is greater than channel capacity for all durations.
1998	Yes	Peak local flow is greater than channel capacity for all durations.
2006	Yes	Peak local flow is greater than channel capacity for all durations.

Baseline evaluation of p=0.005 design events

Table 11 includes simulation results for all durations for the p=0.005 design events scaled using the New Hogan frequency curve. Table 12 includes simulation results for all durations for the p=0.005 design event scaled using the Bellota frequency curve. Figure 26 through Figure 30 show reservoir routings for the 3-day design duration for 5 historical pattern events, scaled to p=0.005 flows using the New Hogan frequency curve. Although not included here, plots for all durations scaled using both New Hogan and Bellota frequency curves are on the CD delivered to the Corps.

The plots show the channel capacity of 12,500 cfs at Bellota is not exceeded for the 1958 and 1986 p=0.005 design events (scaled using either the New Hogan or Bellota frequency curve).

Channel capacity at Bellota is exceeded for all 1997, 1998, and 2006 events (scaled using either the New Hogan or Bellota frequency curve). The channel capacity is exceeded because local flows at Bellota are greater than channel capacity.

ESRD emergency releases are made in the 1998 3- and 4-day events and 2006 6- and 7-day events. ESRD releases are minimum releases that are required to be made to protect the integrity of the dam. The ESRD release is determined by the reservoir inflow and pool elevation. For all simulations which invoked an ESRD release, the release made was greater than channel capacity (12,500 cfs).

The p=0.005 design (scaled) events for the 3-day duration are summarized in Table 13 and Table 14.

Table 11. p=0.005 design events scaled using the New Hogan frequency curve

Event pattern (1)	Duration (days) (2)	Scale factor (3)	Peak regulated flow at Bellota (cfs) (4)	Peak local flow (cfs) (5)	New Hogan peak inflow (cfs) (6)	New Hogan peak release (cfs) (7)
1958	3	1.24	12,500[1]	2,719	62,372	12,466
	4	1.21	12,500[1]	2,654	60,863	12,466
	5	1.19	12,500[1]	2,610	59,857	12,465
	6	1.16	12,500[1]	2,544	58,348	12,465
	7	1.14	12,500[1]	2,500	57,342	12,464
1986	3	1.62	12,500[1]	9,477	57,510	12,352
	4	1.51	12,500[1]	8,834	53,605	12,362
	5	1.51	12,500[1]	8,834	53,605	12,362
	6	1.56	12,500[1]	9,126	55,380	12,357
	7	1.63	12,500[1]	9,536	57,865	12,351
1997	3	2.56	17,659	17,149[2]	64,256	12,500
	4	2.68	18,494	17,953[2]	67,268	12,500
	5	2.78	18,976	18,623[2]	69,778	12,500
	6	2.84	19,509	19,025[2]	71,284	12,500
	7	2.91	19,998	19,494[2]	73,041	12,500
1998	3	3.43	33,892	32,365[2]	86,779	25,651[3]
	4	3.28	31,030	30,950[2]	82,984	22,543[3]
	5	2.92	27,681	27,553[2]	73,876	12,500
	6	2.76	26,208	26,043[2]	69,828	12,500
	7	2.75	26,116	25,949[2]	69,575	12,500
2006	3	2.23	18,171	16,306[2]	61,102	12,500
	4	2.34	18,671	17,110[2]	64,116	12,500
	5	2.44	19,081	17,841[2]	66,856	12,500
	6	2.54	20,829	18,572[2]	69,596	17,872[3]
	7	2.58	23,515	18,865[2]	70,692	19,512[3]

Notes:
1. Reservoir release adjusted by hand to compensate for routing problem in HEC-ResSim. There is sufficient storage to contain event.
2. Local flow is greater than 12,500 cfs.
3. ESRD release.

Table 12. p=0.005 design events scaled using the Bellota frequency curve

Event pattern (1)	Duration (days) (2)	Scale factor (3)	Peak regulated flow at Bellota (cfs) (4)	Peak local flow (cfs) (5)	New Hogan peak inflow (cfs) (6)	New Hogan peak release (cfs) (7)
1958	3	1.33	12,500[1]	2,917	66,899	12,466
	4	1.30	12,500[1]	2,851	65,390	12,465
	5	1.28	12,500[1]	2,807	64,384	12,466
	6	1.25	12,500[1]	2,742	62,875	12,466
	7	1.23	12,500[1]	2,698	61,869	12,465
1986	3	1.52	12,500[1]	8,892	53,960	12,361
	4	1.41	12,500[1]	8,249	50,055	12,372
	5	1.41	12,500[1]	8,249	50,055	12,372
	6	1.46	12,500[1]	8,541	51,830	12,367
	7	1.52	12,500[1]	8,892	53,960	12,361
1997	3	2.42	16,908	16,186[2]	60,742	12,500[1]
	4	2.55	17,605	17,055[2]	64,005	12,500[1]
	5	2.63	18,106	17,590[2]	66,013	12,500[1]
	6	2.70	18,601	18,058[2]	67,770	12,500[1]
	7	2.77	18,914	18,527[2]	69,527	12,500[1]
1998	3	2.98	28,234	28,119[2]	75,394	12,923[3]
	4	2.86	27,127	26,987[2]	72,358	12,500[1]
	5	2.61	24,831	24,628[2]	66,033	12,500[1]
	6	2.51	23,912	23,684[2]	63,503	12,500[1]
	7	2.51	23,912	23,684[2]	63,503	12,500[1]
2006	3	2.11	16,520	15,427[2]	57,814	12,500[1]
	4	2.23	18,171	16,305[2]	61,102	12,500[1]
	5	2.36	18,659	17,255[2]	64,664	12,500[1]
	6	2.41	18,970	17,621[2]	66,034	12,500[1]
	7	2.46	19,140	17,986[2]	67,404	13,309[3]

Notes:
1. Reservoir release adjusted by hand to improve HEC-ResSim routing. There is sufficient storage to contain event.
2. Local flow is greater than 12,500 cfs.
3. ESRD release.

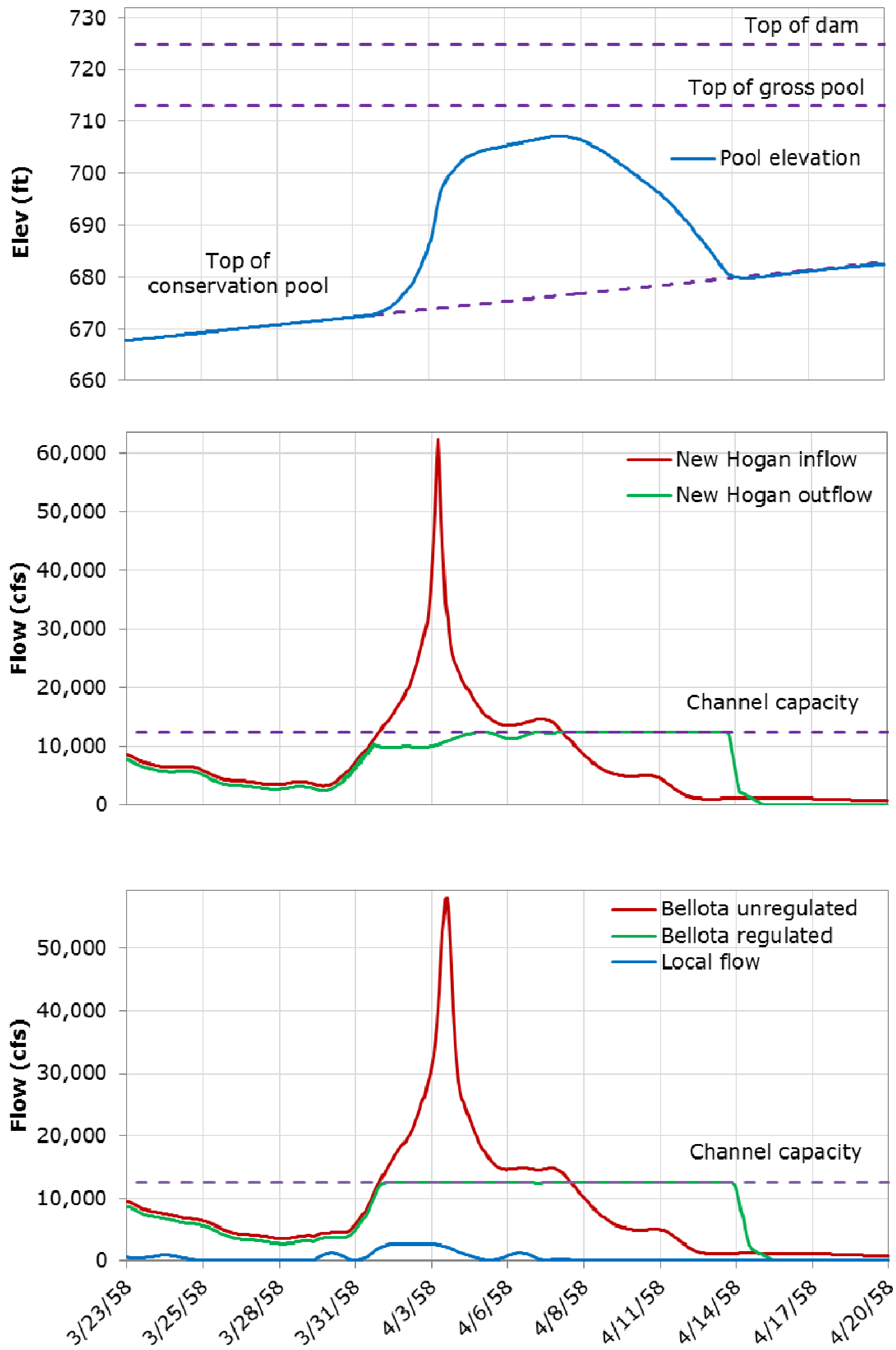


Figure 26. Reservoir routings of the 1958 event scaled using the New Hogan frequency curve to the p=0.005 3-day flow

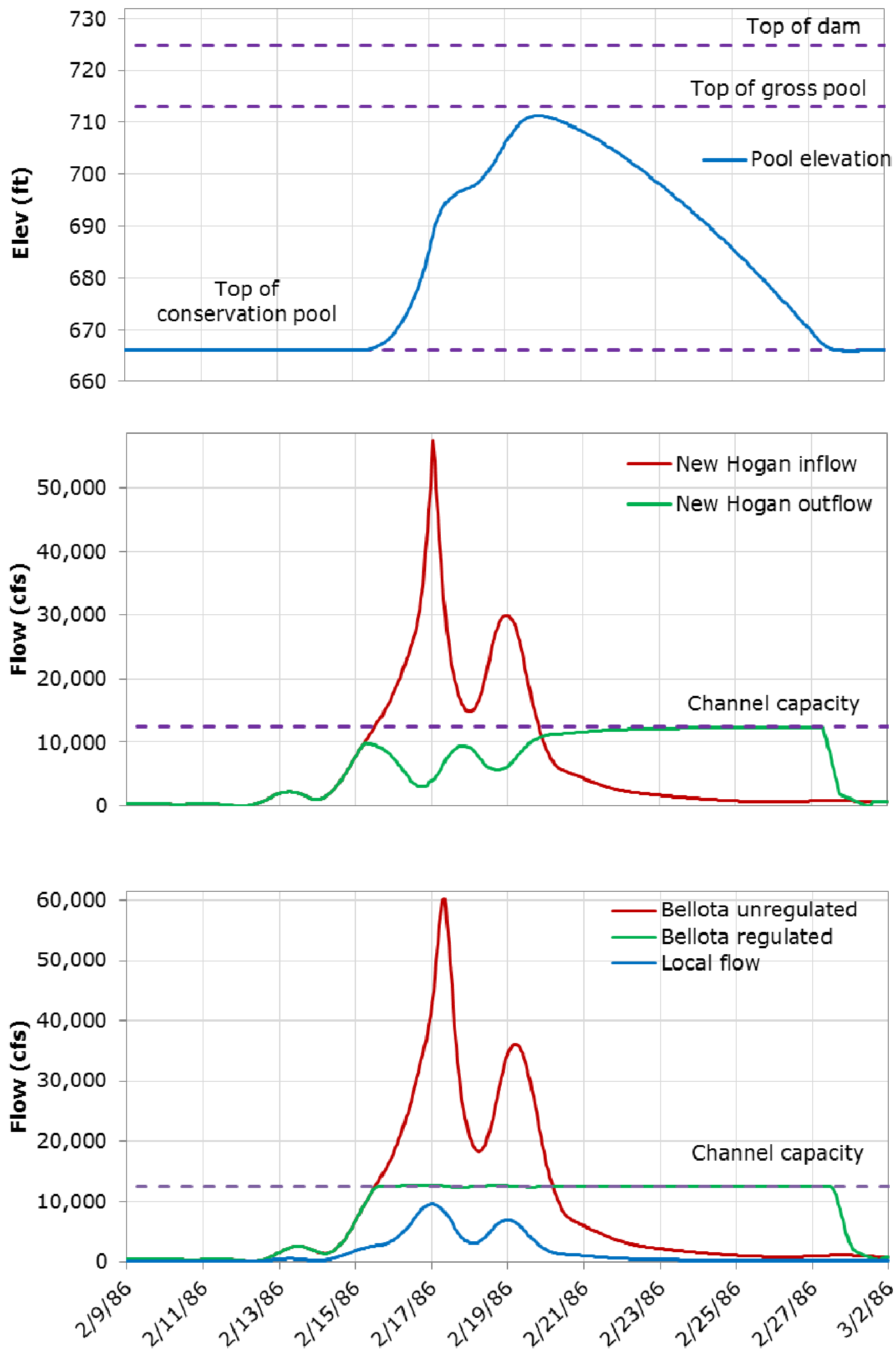


Figure 27. Reservoir routings of the 1986 event scaled using the New Hogan frequency curve to the p=0.005 3-day flow

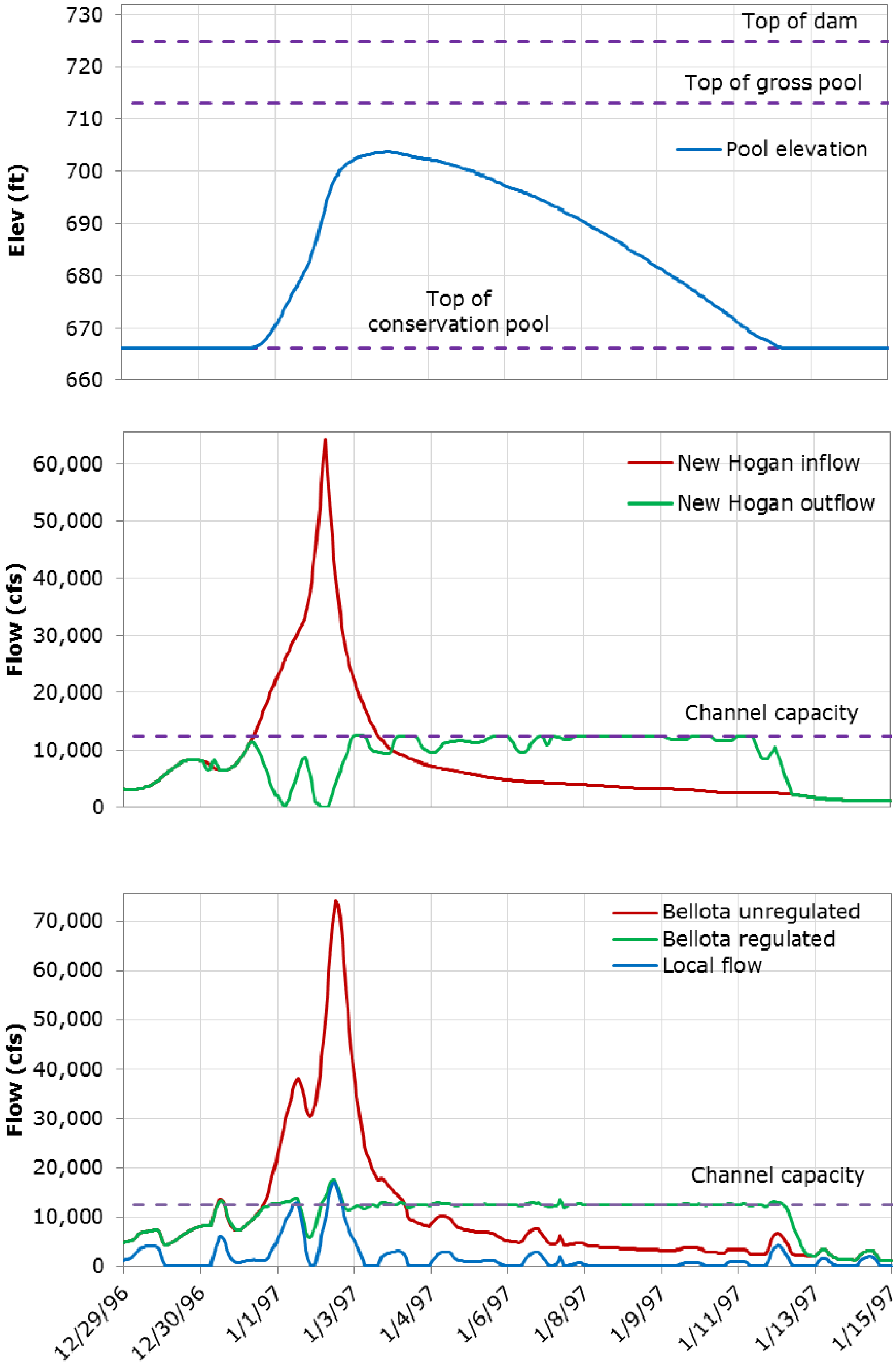


Figure 28. Reservoir routings of the 1997 event scaled using the New Hogan frequency curve to the p=0.005 3-day flow

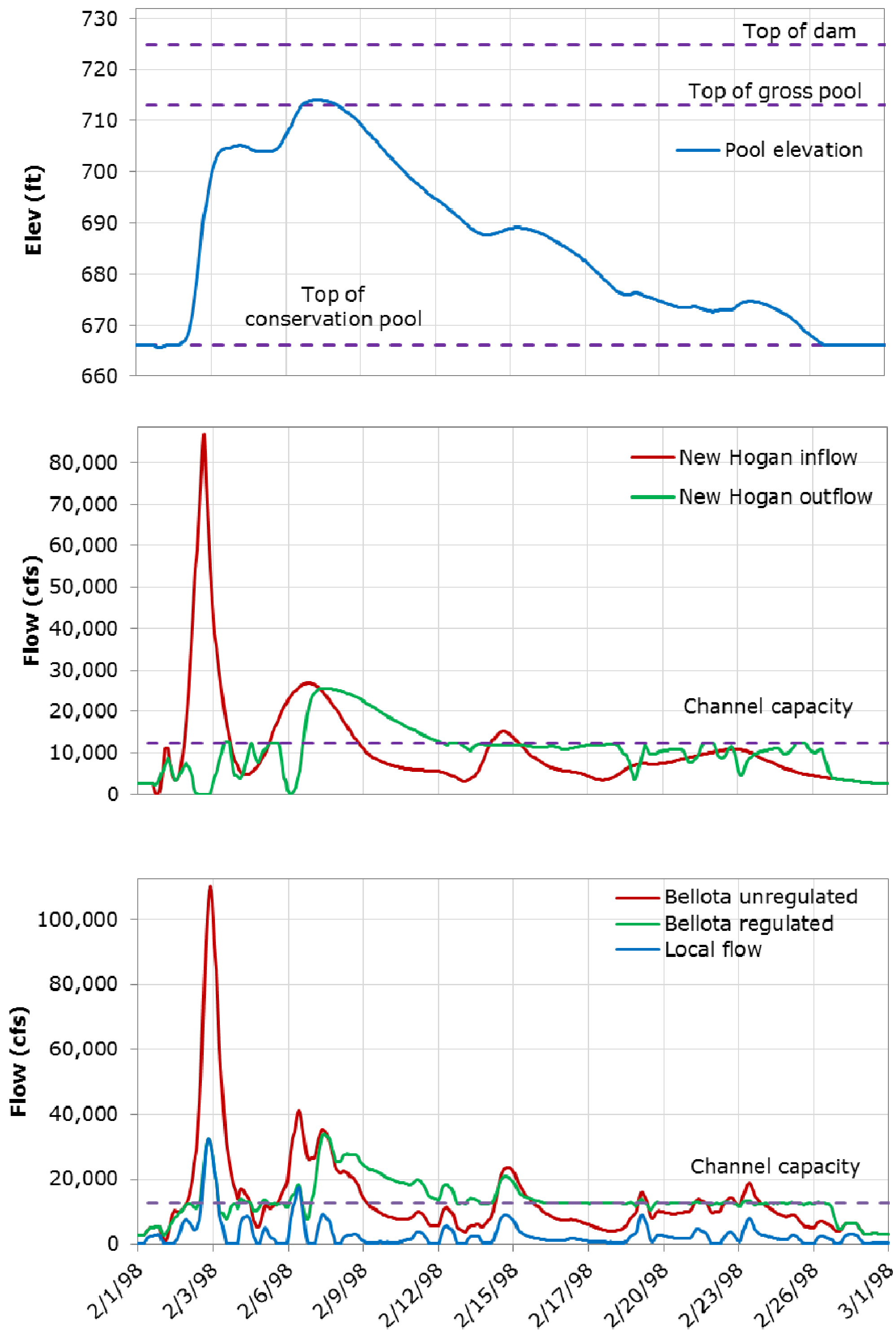


Figure 29. Reservoir routings of the 1998 event scaled using the New Hogan frequency curve to the p=0.005 3-day flow

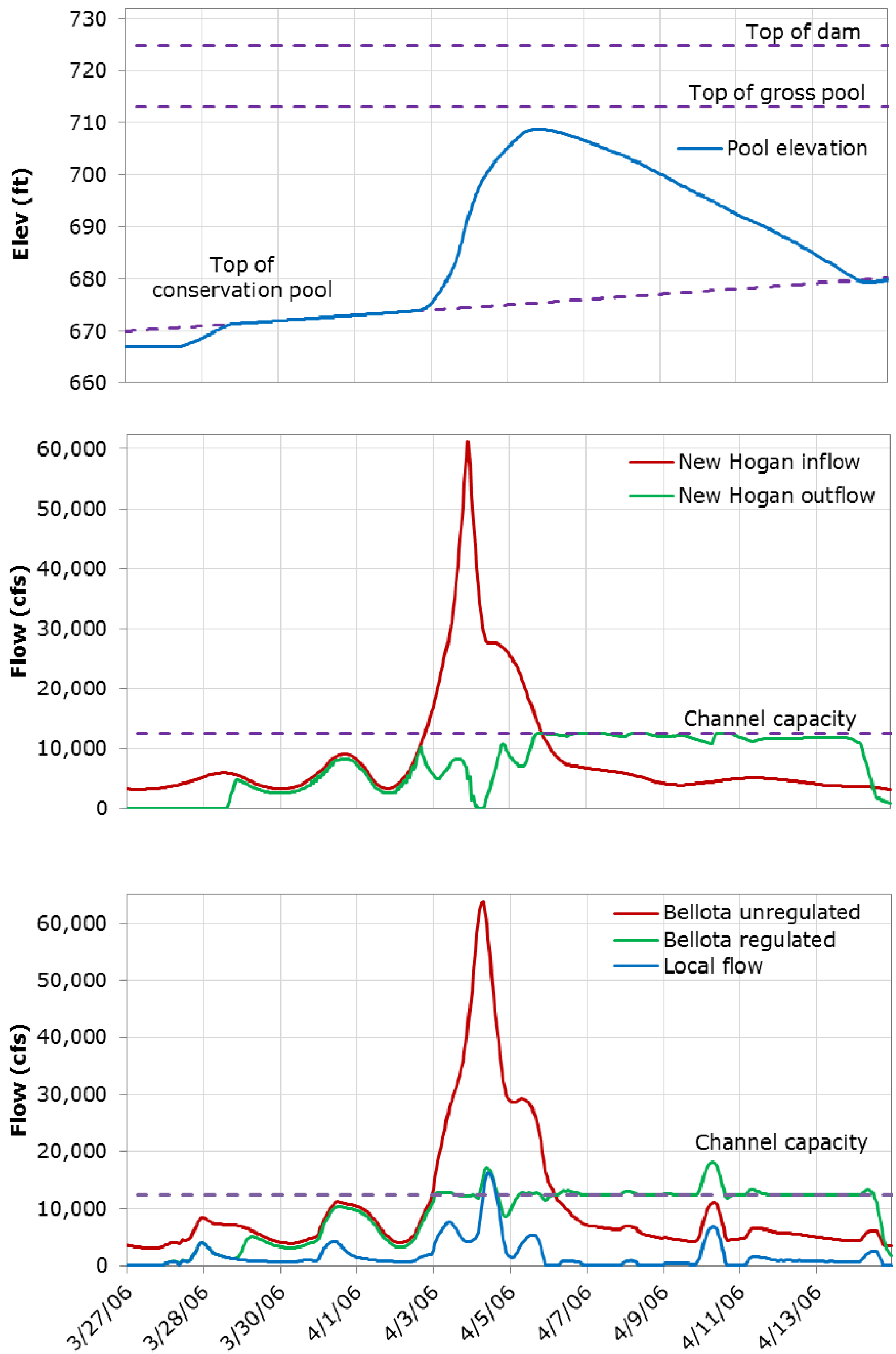


Figure 30. Reservoir routings of the 2006 event scaled using the New Hogan frequency curve to the p=0.005 3-day flow

Table 13. Summary of simulation results for all events analyzed; all flows scaled to the 3-day p=0.005 flow using the New Hogan frequency curve

Pattern event (1)	Channel capacity at Bellota exceeded? (2)	Notes (3)
1958	No	—
1986	No	—
1997	No	—
1998	Yes	Channel capacity at Bellota is exceeded due to local flows for all durations. In addition, ESRD releases are made at the 3-day and 4-day durations.
2006	Yes	Channel capacity at Bellota is exceeded due to local flows. In addition, ESRD releases are made at the 6-day and 7-day durations.

Table 14. Summary of simulation results for all events analyzed; all flows scaled to the p=0.005 flow using the Bellota frequency curve

Pattern event (1)	Channel capacity at Bellota exceeded? (2)	Notes (3)
1958	No	—
1986	No	—
1997	Yes	Channel capacity at Bellota is exceeded due to local flows for all durations.
1998	Yes	Channel capacity at Bellota is exceeded due to local flows for all durations. In addition, ESRD releases are made at the 3-day duration.
2006	Yes	Channel capacity at Bellota is exceeded due to local flows. In addition, ESRD releases are made at the 7-day duration.

Baseline evaluation of p=0.002 design events

Table 15 includes simulation results for all durations for the p=0.002 design events scaled using the New Hogan frequency curve. Table 16 includes simulation results for all durations for the p=0.002 design events scaled using the Bellota frequency curve. Figure 31 through Figure 35 show reservoir routings for the 3-day design duration for 5 historical pattern events, scaled to p=0.002 flows using the New Hogan frequency curve. Although not included here, plots for all durations using the New Hogan frequency curve and the Bellota frequency curve are on the CD delivered to the Corps.

The plots show that channel capacity at Bellota for each of the 5 design hydrographs scaled to the p=0.002 flow using the New Hogan frequency curve is exceeded for all design events except for 3: the 7-day 1958 event scaled to the New Hogan frequency curve, and the 1986 4-day and 5-day events scaled to the Bellota frequency curve. Local flows alone are greater than channel capacity for all 1997, 1998, and 2006 design events. ESRD releases are made for most p=0.002 design events which contribute to downstream flooding.

The p=0.002 design (scaled) events for the 3-day duration are summarized in Table 17 and Table 18.

Table 15. p=0.002 design events scaled using the New Hogan frequency curve

Event pattern (1)	**Duration (days) (2)**	**Scale factor (3)**	**Peak regulated flow at Bellota (cfs) (4)**	**Peak local flow (cfs) (5)**	**New Hogan peak inflow (cfs) (6)**	**New Hogan peak release (cfs) (7)**
1958	3	1.42	16,999	3,114	71,426	16,789[3]
	4	1.40	16,759	3,070	70,420	16,552[3]
	5	1.39	16,628	3,048	69,917	16,422[3]
	6	1.36	15,230	2,983	68,408	15,069[3]
	7	1.33	12,500[1]	2,917	66,899	12,466
1986	3	1.87	35,436	10,940	66,385	31,171[3]
	4	1.75	24,134	10,238	62,125	21,256[3]
	5	1.75	24,134	10,238	62,125	21,256[3]
	6	1.82	30,740	10,647	64,610	27,272[3]
	7	1.90	37,637	11,115	67,450	32,835[3]
1997	3	2.95	20,218	19,762[2]	74,045	12,500
	4	3.09	21,099	20,700[2]	77,559	12,500
	5	3.23	22,642	21,638[2]	81,073	18,817[3]
	6	3.31	24,752	22,174[2]	83,081	20,985[3]
	7	3.40	26,763	22,777[2]	85,340	23,115[3]
1998	3	3.94	41,261	37,178[2]	99,682	30,731[3]
	4	3.80	39,713	35,857[2]	96,140	29,559[3]
	5	3.39	32,726	31,988[2]	85,767	24,706[3]
	6	3.21	30,376	30,290[2]	81,213	21,041[3]
	7	3.21	30,376	30,290[2]	81,213	21,041[3]
2006	3	2.56	22,110	18,719[2]	70,144	18,655[3]
	4	2.70	29,668	19,742[2]	73,980	23,752[3]
	5	2.84	36,275	20,766[2]	77,816	29,521[3]
	6	2.96	39,697	21,644[2]	81,104	32,586[3]
	7	3.01	41,060	22,009[2]	82,474	33,868[3]

Notes:
1. Reservoir release adjusted by hand to improve HEC-ResSim routing. There is sufficient storage to contain event.
2. Local flow is greater than 12,500 cfs.
3. ESRD release.

Table 16. p=0.002 design events scaled using the Bellota frequency curve

Pattern event (1)	Duration (days) (2)	Scale factor (3)	Peak regulated flow at Bellota (cfs) (4)	Peak local flow (cfs) (5)	New Hogan peak inflow (cfs) (6)	New Hogan peak release (cfs) (7)
1958	3	1.52	21,409	3,334	76,456	20,638[3]
	4	1.49	19,897	3,268	74,947	18,799[3]
	5	1.47	18,738	3,224	73,941	17,458[3]
	6	1.45	17,620	3,180	72,935	17,018[3]
	7	1.42	16,999	3,114	71,426	16,789[3]
1986	3	1.74	23,159	10,179	61,770	20,414[3]
	4	1.62	12,500[1]	9,477	57,510	12,352
	5	1.63	12,500[1]	9,536	57,865	12,351
	6	1.69	17,338	9,887	59,995	15,171[3]
	7	1.76	25,065	10,296	62,480	22,080[3]
1997	3	2.76	18,852	18,460[2]	69,276	12,500[1]
	4	2.93	20,109	19,597[2]	73,543	12,500[1]
	5	3.03	20,892	20,266[2]	76,053	12,500[1]
	6	3.12	21,194	20,867[2]	78,312	12,500[1]
	7	3.21	21,745	21,469[2]	80,571	17,854[3]
1998	3	3.40	33,019	32,082[2]	86,020	24,976[3]
	4	3.29	31,123	31,044[2]	83,237	22,770[3]
	5	3.00	28,419	28,308[2]	75,900	14,166[3]
	6	2.90	27,496	27,364[2]	73,370	12,500[1]
	7	2.91	27,589	27,459[2]	73,623	12,500[1]
2006	3	2.41	18,970	17,621[2]	66,034	12,500[1]
	4	2.56	22,110	18,717[2]	70,144	18,655[3]
	5	2.72	30,654	19,887[2]	74,528	24,499[3]
	6	2.79	34,307	20,399[2]	76,446	27,842[3]
	7	2.85	36,596	20,838[2]	78,090	29,784[3]

Notes:
1. Reservoir release adjusted by hand to improve HEC-ResSim routing. There is sufficient storage to contain event.
2. Local flow is greater than 12,500 cfs.
3. ESRD release.

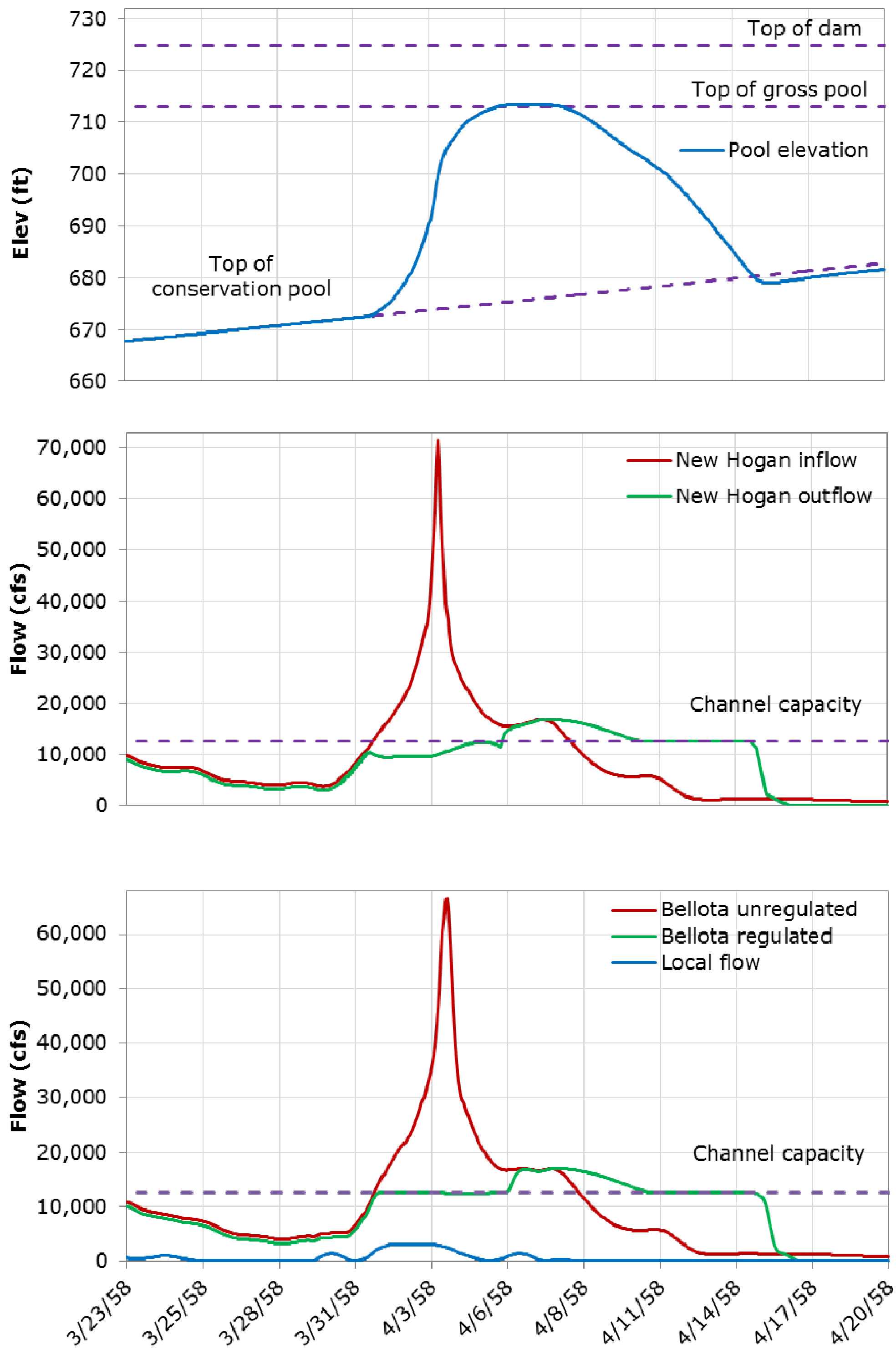


Figure 31. Reservoir routings of the 1958 event scaled using the New Hogan frequency curve to the p=0.002 3-day flow

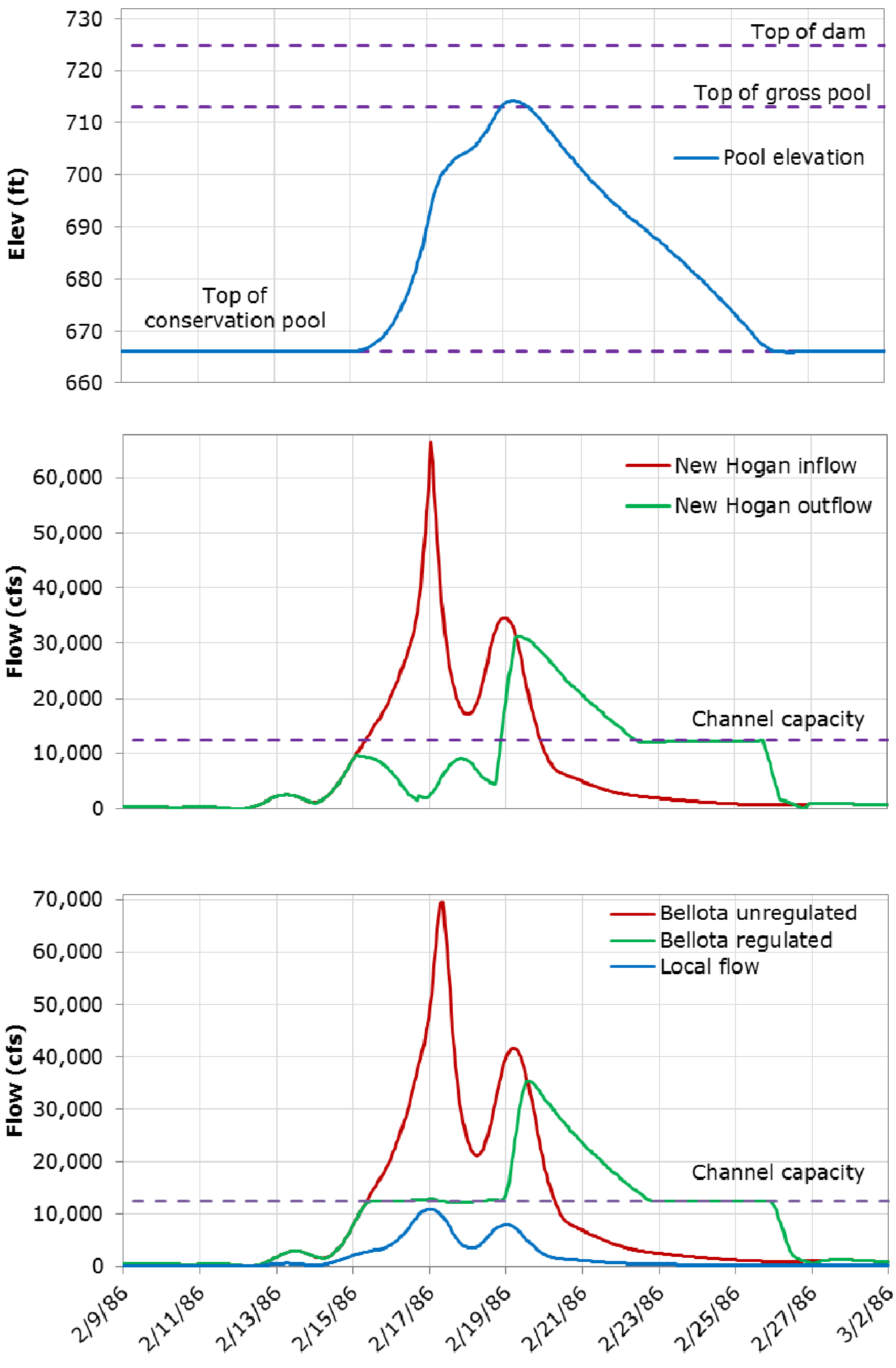


Figure 32. Reservoir routings of the 1986 event scaled using the New Hogan frequency curve to the p=0.002 3-day flow

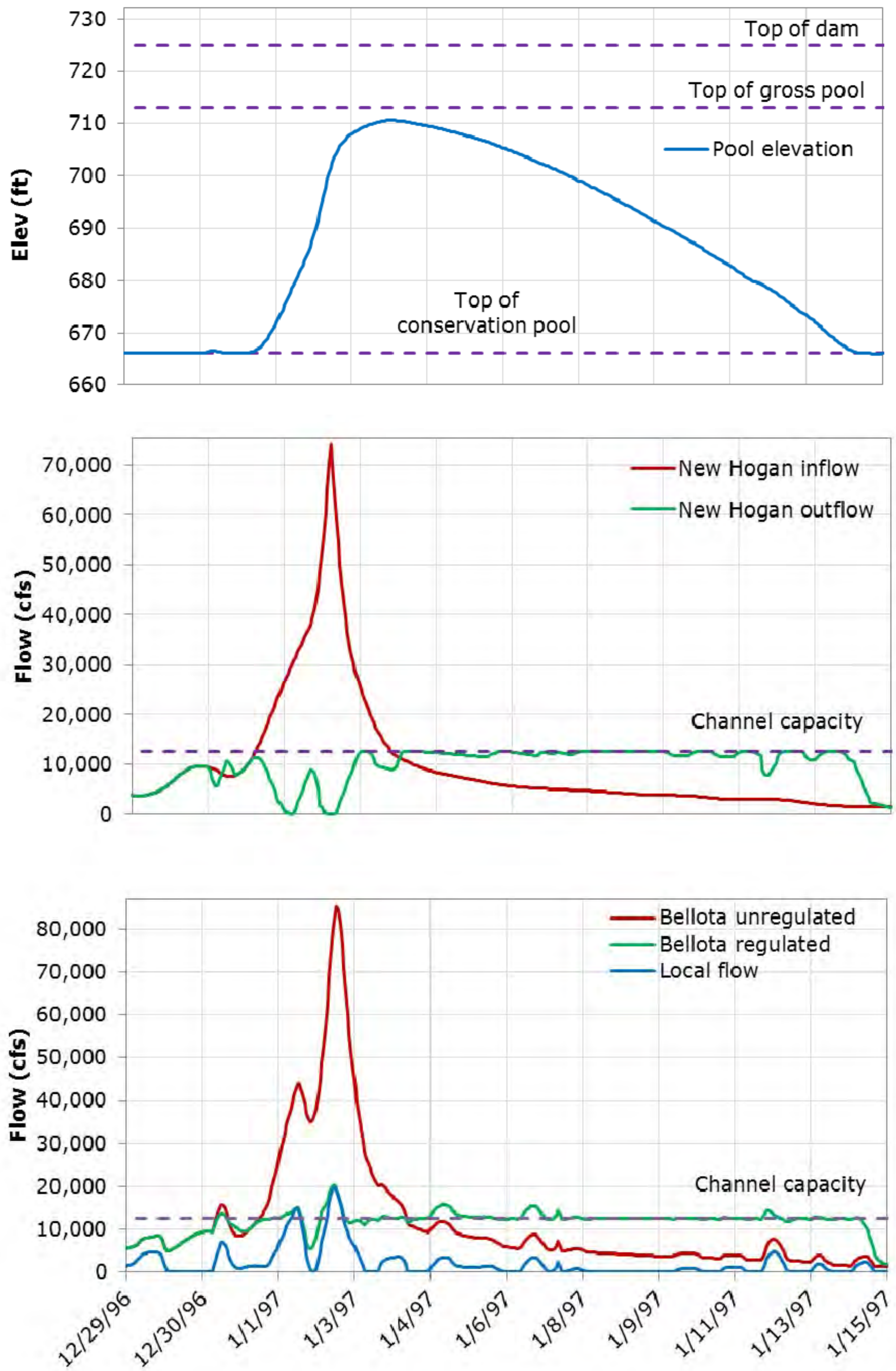


Figure 33. Reservoir routings of the 1997 event scaled using the New Hogan frequency curve to the p=0.002 3-day flow

Figure 34. Reservoir routings of the 1998 event scaled using the New Hogan frequency curve to the p=0.002 3-day flow

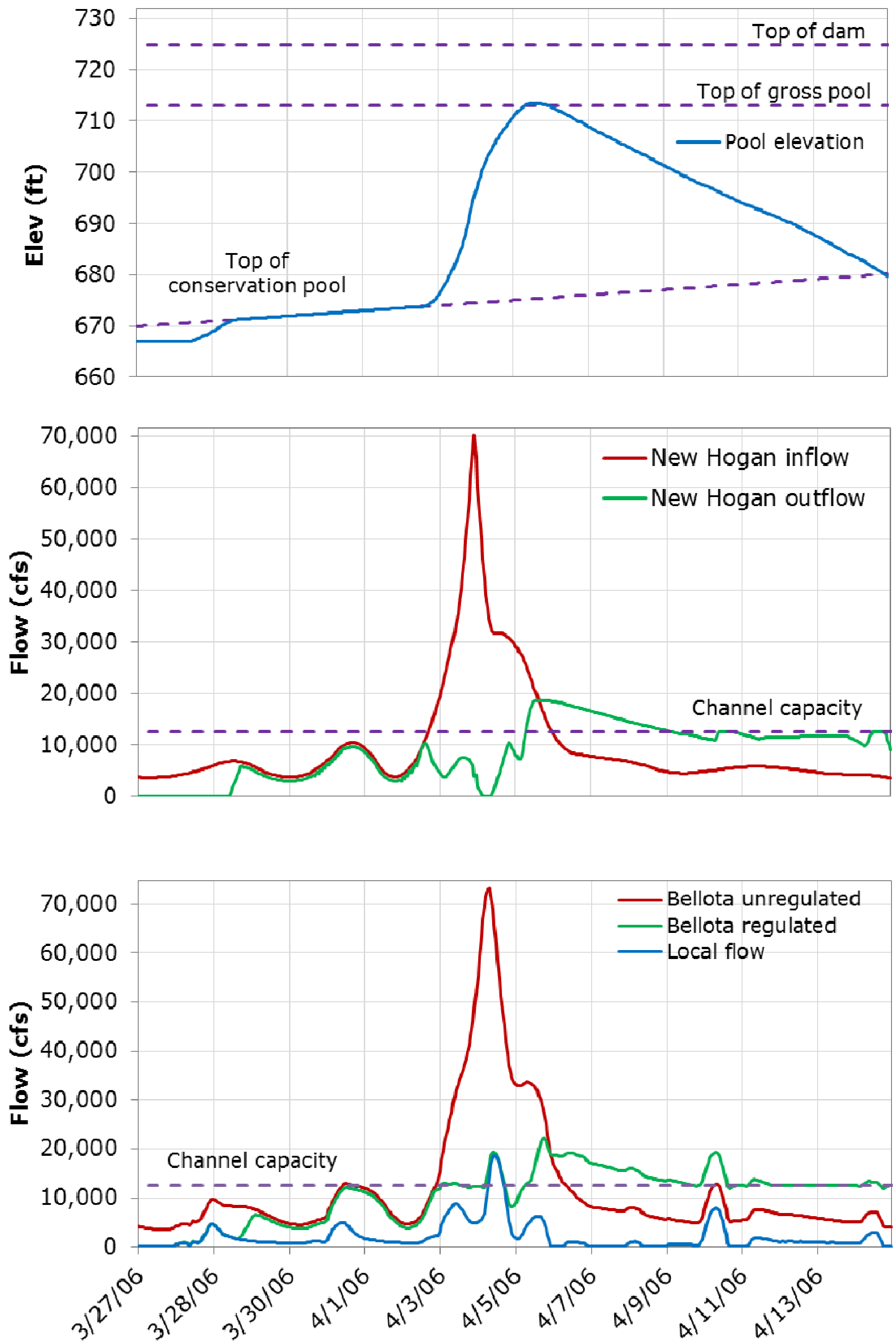


Figure 35. Reservoir routings of the 2006 event scaled using the New Hogan frequency curve to the p=0.002 3-day flow

Table 17. Summary of New Hogan operation design events; all flows scaled to p=0.002 flows using the New Hogan frequency curve

Pattern event (1)	Channel capacity at Bellota exceeded? (2)	Notes (3)
1958	3-,4-,5-,6-day: Yes 7-day: No	ESRD releases are made at all durations except for the 7-day.
1986	Yes	ESRD releases are made at all durations.
1997	Yes	Peak local flow exceeds channel capacity at all durations. In addition, ESRD releases are made at 3-day, 6-day, and 7-day durations.
1998	Yes	Peak local flow exceeds channel capacity at all durations. In addition, ESRD releases are made at all durations.
2006	Yes	Peak local flow exceeds channel capacity at all durations. In addition, ESRD releases are made at all durations.

Table 18. Summary of regulated flows at Bellota for design events scaled to p=0.002 flows using the Bellota frequency curve

Pattern event (1)	Channel capacity at Bellota exceeded? (2)	Notes (3)
1958	Yes	ESRD releases are made at all durations.
1986	3-,6-,7-day: Yes 4-,5-day: No	ESRD releases are made at the 3-day, 6-day, and 7-day durations.
1997	Yes	Peak local flow exceeds channel capacity at all durations. In addition, ESRD releases are made at the 7-day duration.
1998	Yes	Peak local flow exceeds channel capacity at all durations, In addition, ESRD releases are made at the 3-day, 4-day and 5-day durations.
2006	Yes	Peak local flow exceeds channel capacity at all durations. In addition, ESRD releases are made at the 4-day, 5-day, 6-day, and 7-day durations.

Attachment D. Memorandum of study plan

The following alternative analysis plan was provided to Corps staff June 17, 2011.

New Hogan Reservoir re-operation sensitivity analysis summary

Task 6 and Option Task 1 of our current scope of work calls for completion of initial sensitivity analysis regarding New Hogan Reservoir re-operation alternatives for containing p=0.005 flows at Bellota. The current channel capacity is reported to be 12,500 cfs.

Our scope of work describes the required analysis and the specific questions that we will answer. The scope of work does call for providing a technical memorandum to identify which historical or scaled historical events we will use for the analysis.

Also included in the scope of work is an assessment of how the regulated flow for the selected events would change with an increased channel capacity at Bellota.

In this memorandum, we:

- Describe the reservoir simulation model we will use for the analysis.
- Propose selected historical and scaled historical events for both the re-operation simulations. These are the same events that will be used for Task 6 and Option Task 1.

Reservoir simulation model: HEC-ResSim

For the reservoir simulations, we will use computer program HEC-ResSim. Specifically, we will use the model of New Hogan used in the Lower San Joaquin River (LSJR) feasibility study provided to the Corps in June 2011. This study used Version 3.1 Build 101.

A known computational bug exists in this version regarding the reservoir operation for downstream constraints when Muskingum routing is used. The downstream channel capacity may be exceeded due to this bug, even when there is sufficient storage in the flood pool to contain the event. If we notice this problem in any of the simulations, we will make a note of our findings and inform the SPK technical lead. Further, we will evaluate the simulation results when the channel capacities are exceeded as to if additional flood storage is available in New Hogan Reservoir or not. We will not complete hand routings or use "release overrides" to correct the computer program simulations.

Regarding the application of HEC-ResSim for this analysis, we will:

- For the increased reservoir storage analysis, configure the model to increase the storage by lowering the flood pool. We will simulate selected events through a series of trials to determine the minimum amount of flood storage required to meet the downstream channel capacity of 12,500 cfs at Bellota.

- For all simulations, we will ensure that the current New Hogan Dam outlet works do not limit the release capacity from the dam. (If release capacity is an issue, we will note this.)
- For all simulations, keep the rate of change and ESRD operation rules in the model.
- For all simulations, when the downstream objective flow is exceeded, we will evaluate the simulation and identify the limiting rule or constraint and note this.

Selected historical and scaled historical events

The time series inputs for this analysis will be the same as those used for our June 2011 baseline analysis. This includes both the reservoir inflow and the corresponding local flow between New Hogan and Bellota.

For the event selection, we used the following considerations for selecting events:

- Regulated peak flow close to the p=0.005 peak flow at Bellota from the June 2011 study, which is 16,407 cfs.
- Preference given to events with low scale factors.
- Preference given to events that have local flows developed based on hourly observed values.

Further, in finalizing the selection, we chose at least 3 events for which the local flow at Bellota is less than the channel capacity of 12,500 cfs, and chose events that had showed a ranged of shapes (temporal distribution.)

The 7 events that best matched this above criteria are those shown in Table 19. We will select a minimum of 5 events from this table for use in the analysis. If needed based on the simulations and any errors we find in the reservoir simulations, we may use the remaining events in Table 19.

Proposed increased channel capacity

For Option Task 1, we will simulate the selected historical or scaled historical events for 1 alternative channel capacity at Bellota. Alternative capacities proposed by Dave Peterson, Peterson, Brustad, Inc. in a memorandum to us dated November 29, 2010 are 15,000 cfs, 18,000 cfs, and 21,000 cfs. The increased channel capacity we will use is to be decided upon by the project team. For each simulation, we will report the change in peak release from the reservoir and the peak regulated flow at Bellota.

Table 19. Candidate historical and scaled historical events for analysis

Event (1)	**Scale factor (2)**	**Hourly local flows? (3)**
Criteria	1.0	Yes
1997	2.2	Yes
1958	1.4	No
1986	1.6	Yes
1907	2.2	No
1998	1.6	Yes
1999	1.0	Yes
2006	1.0	Yes

Attachment E. Evaluation of New Hogan re-operation alternative with selected events

Overview

The June 2011 results, as shown in Table 1, show that the 12,500 cfs channel capacity at Bellota is exceeded for the p=0.01 and p=0.005 events. One of the flood risk reduction measures being considered by the study team is the increased flood storage in New Hogan Reservoir. Specifically, the question is how much additional flood storage capacity is needed to contain the p=0.005 peak flow at Bellota within the existing channel capacity.

Volume analysis

Before completing reservoir routings of design (pattern) events, we completed a volume analysis based on the reservoir inflow-frequency curves and the available flood storage in New Hogan Reservoir.

Using the unregulated flow-frequency curve in our June 2011 report, included as Figure 9 in that report, we tabulated the volume for various flow quantiles and durations. Table 20 lists average flows for the p=0.01 1-, 3-, 4-, 5-, and 7-day durations from the frequency curve in column 2. In column 3, we convert the values from column 2 from an average flow for a specified duration to a total volume for the same duration. Table 21 is a similar table, but uses the p=0.005 flows from the frequency curve.

Table 20. Volume analysis for the p=0.01 event using the June 2011 inflow-frequency curve

Duration (days) (1)	Average flow for specified duration and AEP (cfs) (2)	Total volume for specified duration (ac-ft) (3)
1	36,000	71,500
3	24,400	145,000
4	21,100	167,500
5	18,900	187,300
7	16,000	221,700

Table 21. Volume analysis for the p=0.005 event using the June 2011 inflow-frequency curve

Duration (days) (1)	Average flow for specified duration and AEP (cfs) (2)	Total volume for specified duration (ac-ft) (3)
1	40,700	80,700
3	27,700	165,000
4	24,100	191,300
5	21,600	214,600
7	18,400	255,100

The current flood control storage in New Hogan Reservoir is 165,000 ac-ft between November 30 and March 20 per the water control manual (USACE 2004).

Comparing the runoff volumes in column 3 of Table 20**Error! Reference source not found.** and Table 21, we find:

- For durations of approximately 4 days or less for the p=0.01 flows, the entire runoff volume can be stored within the designated flood storage.
- For the durations of approximately 3 days or less for the p=0.005 flows, the entire runoff volume can be stored within the designated flood storage.

Thus, this implies that such an event should be able to be contained within the reservoir with no release. However, in reality, events are longer than 3 to 4 days. Further, the downstream local flows do not fill the entire channel capacity for that long either, thus the reservoir does not need to stop all releases.

Reservoir simulations and alternative analysis

Using the events from Table 2, we evaluated the impact of increased storage. As shown in Table 3, the only event where additional storage would help reduce flooding at Bellota is the 1958 event scaled by 1.4. Simulation results for this event show that with current storage, the flood pool was full, and emergency releases were made which contributed to downstream flooding at Bellota. Only this event was used for this analysis as additional storage would not help reduce downstream peak flows for the other events listed in the table.

Increased storage was simulated by shifting storage from the conservation pool by lowering the flood pool elevation (as opposed to increasing flood storage by raising the dam), such that the p=0.005 flow is within the current channel capacity (reservoir operation control) at Bellota. Thus, given the local flow contribution between New Hogan Dam and Bellota, the New Hogan Dam release for the p=0.005 event would be less than 12,500 cfs, the channel capacity at Bellota. We found the minimum additional storage through an iterative process. For the simulations, we used the same HEC-ResSim model as used in the June 2011 analysis.

Table 22 lists the trial simulations for the 1958 event and Figure 36 shows a plot of simulation results for the existing condition and with the minimal amount of additional storage needed so the channel capacity at Bellota is not exceeded.

Findings

Consistent with the findings from the design (scaled) event simulations described in Attachment C, the local flows tend to be the dominant factor for peak flows exceeding downstream channel capacity. Further, the volume analysis described here shows that the existing flood storage in New Hogan Reservoir is greater than the 3-day p=0.005 flow. However, for the 1958 event scaled by 1.4, an additional 14,000 acre-ft of flood storage would help to control downstream flows to within channel capacity.

Table 22. Trial simulations for 1958 event scaled by 1.4

Simulation (1)	**Elevation of bottom of flood pool (ft) (2)**	**Capacity at bottom of flood pool (ac-ft) (3)**	**Additional flood storage (ac-ft) (4)**	**Peak flow at Bellota (cfs) (5)**	**Peak pool elevation (ft) (6)**
Existing condition	666.16	152,105	None	16,759	713.4
Trial 1	660	135,292	16,813	12,500	712.7
Trial 2	661	137,948	14,157	12,500	713.2

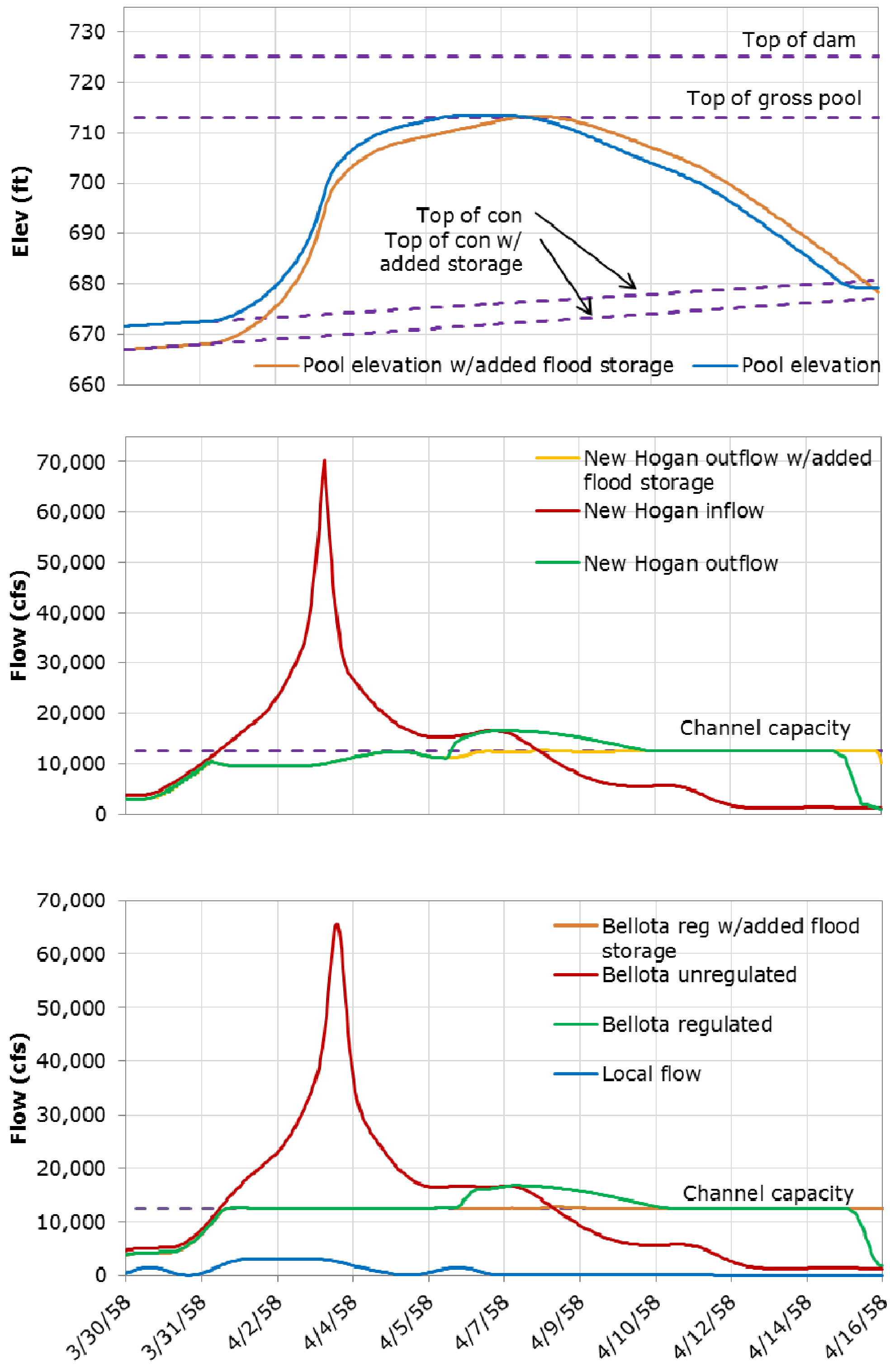


Figure 36. 1958 event scaled to p=0.005, existing condition and with 14,157 ac-ft additional flood storage (created by lowering the top of conservation pool)

Attachment F. Evaluation of channel capacity alternative with selected events

Overview

The June 2011 results, as shown in Table 1, show that the 12,500 cfs channel capacity at Bellota is exceeded for the p=0.01 and p=0.005 events. One of the flood risk reduction measures being considered by the study team is the increased channel capacity. Specifically, the question is how much additional downstream capacity is needed to contain the p=0.005 peak flow.

Analysis

Based on the analysis completed and documented in the previous attachments, we have found that the local flows are the dominant factor in the regulated peak flow-frequency curve at Bellota. For the design (scaled) events using the Bellota unregulated flow-frequency curve, from the June 2011 report, the channel capacity at Bellota was exceeded for the 1997, 1998, and 2006 patterned p=0.005 design (scaled) events. These are shown in Table 12. The channel capacity exceedence for these is due to the local flows, not because of the loss of reservoir flood storage.

To evaluate further these selected design events, we re-simulated the events forcing release to 0 cfs during the period the channel capacity was previously exceeded. These re-simulations are shown in Figure 37, Figure 38, and Figure 39 for the 1997, 1998, and 2006 patterned events respectively. The figures show that the 3-day p=0.005 events can be contained with the current storage in New Hogan and that the flooding at Bellota is due to local flows only. Therefore, the need for increased channel capacity at Bellota is dependent on the local flow-frequency curve.

Findings

Given that for the p=0.005 design (scaled) events, the local flows between New Hogan Reservoir and Bellota drive the peak flow at Bellota, an accepted local flow-frequency curve must be developed and evaluated. Consistent with the guidance in EM 1110-2-1415, as included in Attachment C, the local flow-frequency curve "…is an indicator of the lower limit for the curve of regulated flow."

The limited-use local flow-frequency curve developed herein and included in Attachment B is based on a limited record. Before adoption and acceptance for this purpose, additional analysis is recommended. Further, as part of the LSJR FS, a separate effort is being completed to develop a local flow-frequency curve using rainfall-runoff models and design storms that could also be considered for use for this purpose.

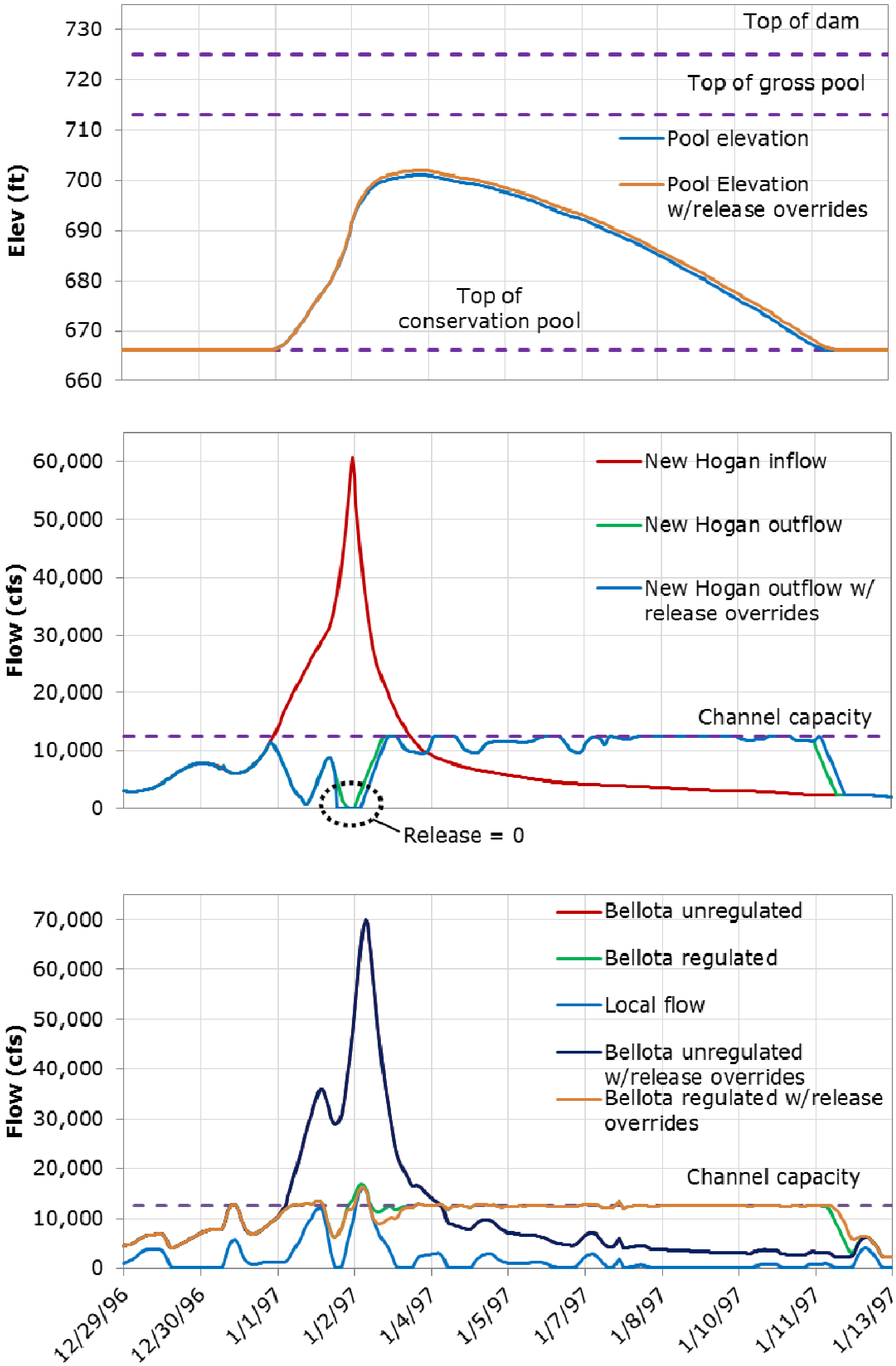


Figure 37. 1997 event scaled to the p=0.005 3-day flow using the Bellota frequency curve; reservoir releases set to 0 cfs during the peak; channel capacity at Bellota is exceeded because of local flows

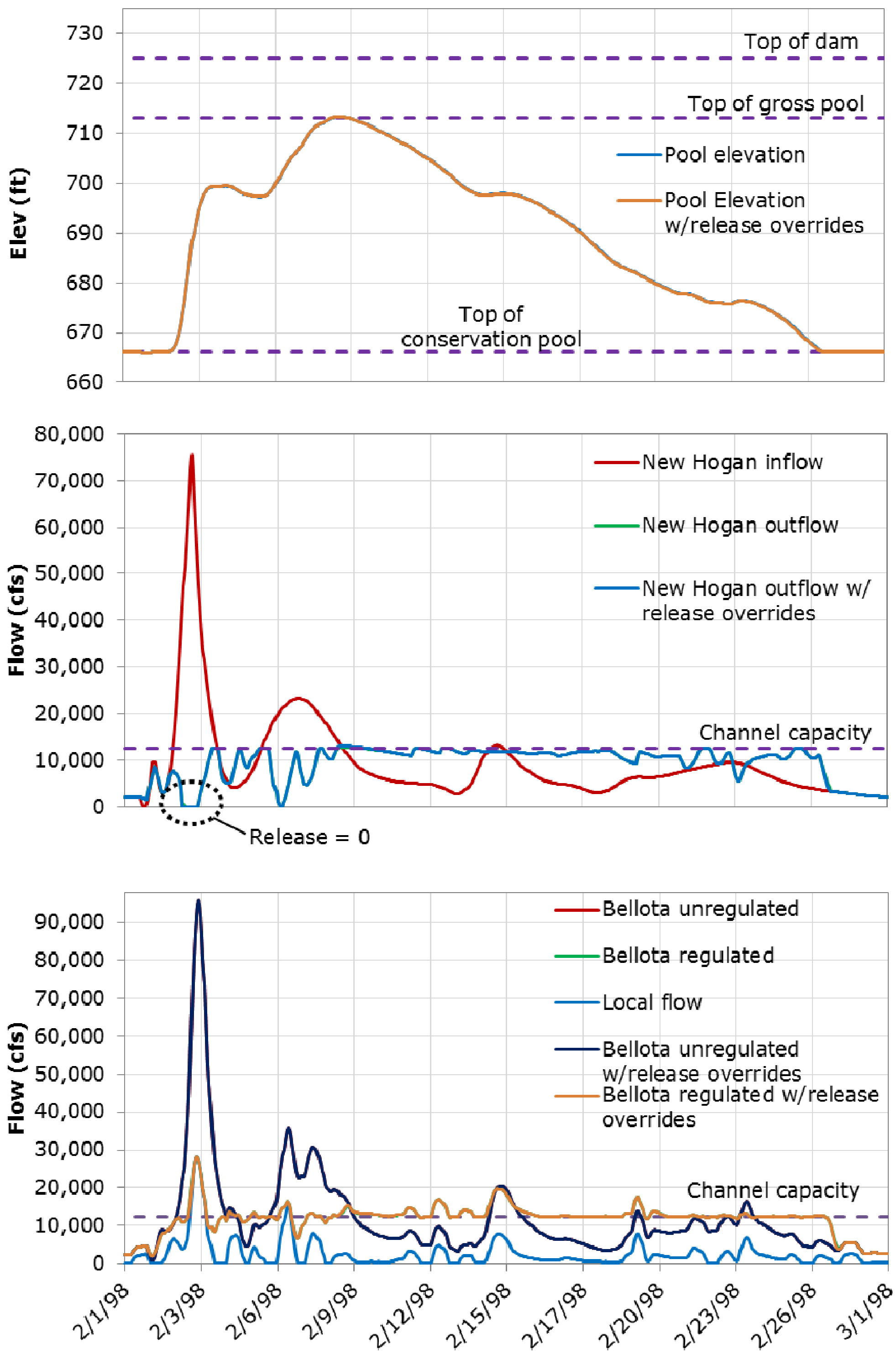


Figure 38. 1998 event scaled to the p=0.005 3-day flow using the Bellota frequency curve; reservoir releases set to 0 cfs during the peak; channel capacity at Bellota is exceeded because of local flows

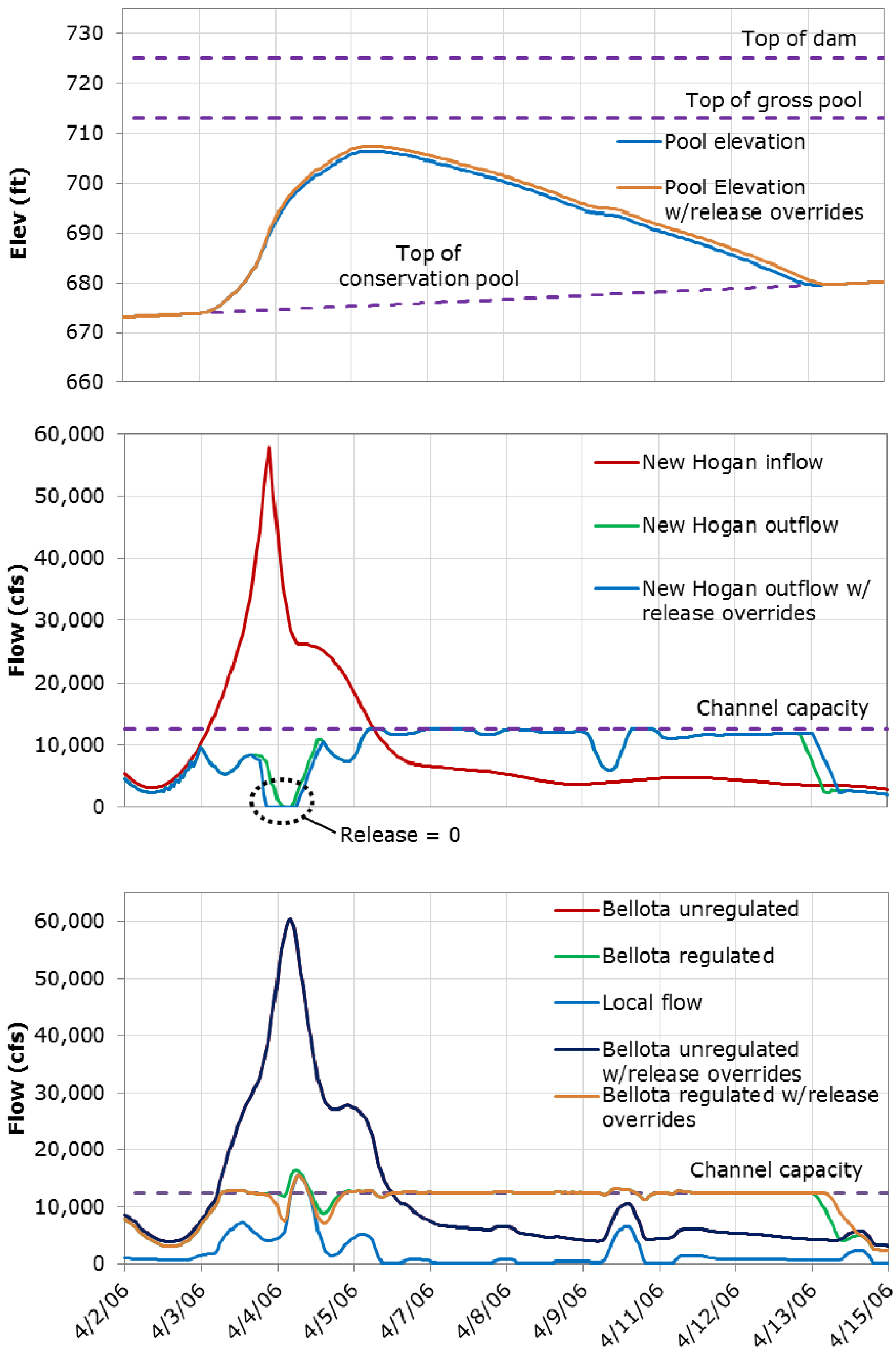


Figure 39. 2006 event scaled to the p=0.005 3-day flow using the Bellota frequency curve; reservoir releases set to 0 cfs during the peak; channel capacity at Bellota is exceeded because of local flows

Attachment G. List of files on CD delivered to the Corps

Table 23 describes the analysis files on the CD delivered to the Corps.

Table 23. Description of files on CD delivered to the Corps

ID (1)	File (2)	Description (3)
1	LSJQMethod2.7zip	HEC-ResSim model and simulations of scaled events
2	LSJQ_Re-opAnalysis.7zip	HEC-ResSim simulations of re-operation analysis
3	NewHogan_re-operation_plan_rev.pdf	Analysis plan
4	Plots.zip	New Hogan and Bellota reservoir routings
5	Scalings.xlsx	Scale factors for all simulations
6	CalaverasRiverLocalFlowFreq.zip	Limited-use local flow-frequency curve input files and program executable

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Appendix 2

Lower San Joaquin River Feasibility Study
Littlejohn Creek above Farmington, Ca.
Hydrologic Analysis

US Army Corps
of Engineers.

Sacramento District

23 June 2014

Contents

List of Tables

List of Figures

Attach 1: **Littlejohn Creek frequency analysis and hydrographs, June 23, 2011**

Background
The Corps of Engineers, Sacramento District, Hydrology Section (SPK) tasked David Ford Consulting Engineers, Inc (DFC) with the derivation of unregulated and regulated flow-frequency curves at Littlejohn Creek at Farmington Dam and Littlejohn Creek at Farmington (main control point for Farmington Dam). Their report is titled: "Lower San Joaquin River feasibility study: Littlejohn Creek frequency analysis and hydrographs" dated June 23, 2011. After DFC performed their analysis, revisions were made by SPK in February of 2012. These include 1) a newer version of HEC-ResSim was utilized for flood routing since the version DFC utilized had difficulty maintaining the objective flow releases downstream – mainly due to local flow fluctuations 2) SPK reduced to four rather than nineteen the number of pattern floods used for scaling and routing through Res-Sim. As floods equal to or exceeding the 1% ACE event are the primary focus of alternatives in this study, SPK used only patterns that were representative of rare floods. The parts of the DFC analysis that remain valid and are incorporated into SPK's adopted hydrology are 1) unregulated frequency curve analyses including derivation of local flows during historic events 2) analysis of the critical duration and 3) the peak to volume characteristic curves. The parts of the DFC report that are superseded include 1) adopted unregulated to regulated peak flow transform and final regulated peak flow frequency curves at each index point. The DFC Report is attached to this Appendix and superseded sections have red watermarks labeled as such. The SPK report describes the final adopted hydrology for the feasibility study.

The lower watershed downstream of the Farmington gage was analyzed by Petersen Brustad, Inc (PBI) using a rainfall runoff model. See Appendix 3 for details on that analysis. The various frequency hydrographs developed at the Farmington gage by SPK (as described in this chapter) became boundary condition input to the HMS model of the French Camp Slough produced by PBI. One of the major purposes of the HMS model was to produce concurrent local flow hydrographs for areas downstream of Farmington, during a specific ACE flood event occurring at the Farmington gage.

It should be noted that an unregulated flow frequency curve at Farmington was the foundation for derivation of a regulated flow frequency curve at the Farmington gage control point. As such, the adopted regulated quantile flows are based on many different storm centerings that the gage has encountered during its long period of record.

The study area for the upper Littlejohn Creek watershed is shown in figure 1 below.

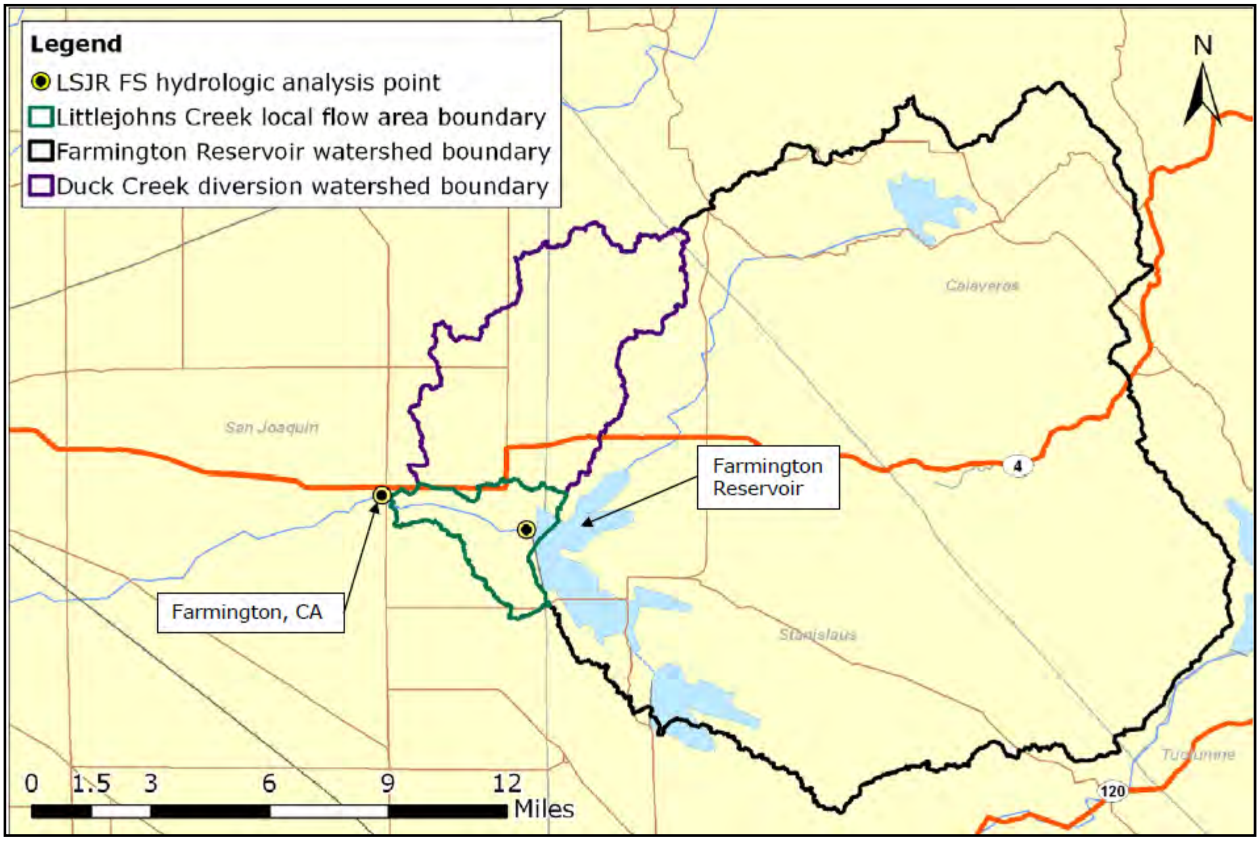


Figure 1. Upper Littlejohn Creek Study area

2.0 Watershed description

The watershed that is the subject of this report—Littlejohn Creek basin—is part of the lower San Joaquin River basin. It is located in Calaveras, San Joaquin, and Stanislaus counties. Located on Littlejohn Creek approximately 20 miles upstream of Stockton, CA, is Farmington Reservoir, a "dry dam" whose primary purpose is flood control.

The principal feature of the watershed is Farmington Reservoir, which drains approximately 212 mi^2. The watershed above the reservoir is wing-shaped and extends 20 miles upstream into the foothills of the western Sierra Nevada. Elevations range from approximately 2,600 ft to approximately 115 ft at the dam.

In addition to runoff from the foothills, Farmington Reservoir receives flows from a diversion on the Stanislaus River at Goodwin Dam, the Stockton East Tunnel, and the Farmington-Stockton East Canal. These flows occur primarily during the summer months and not during the flood season, typically defined as October 1 to May 1 of each water year.

Downstream of Farmington Dam, approximately 3.5 miles, is the Duck Creek Diversion, which diverts flow into Littlejohn Creek from Duck Creek above the town of Farmington. The watershed above the diversion structure on Duck Creek is approximately 28 mi^2. The channel capacity of Duck Creek below the diversion structure is 700 cfs, and the diversion structure itself has a peak capacity of 500 cfs. In addition, the confluence of Littlejohn Creek and Rock Creek is approximately 2 miles downstream of Farmington Dam.

From the town of Farmington, Littlejohn Creek continues west, splitting into the North Fork Littlejohn Creek and South Fork Littlejohn Creek. Flow finally joins French Camp Slough before continuing on to the San Joaquin River. The confluence of Littlejohn Creek and French Camp Slough is located approximately 25 miles downstream of Farmington Dam.

Farmington Reservoir operates to maintain peak flows below the downstream channel capacity of 2,000 cfs near the town of Farmington, including anticipated coincident flows from the Duck Creek Diversion (USACE 2004).

3.0 Procedure for Analysis

The following steps were used to derive hydrographs for Littlejohn Creek at Farmington.

- Develop unregulated flow time series including Farmington Dam inflow and local flow (between dam and the Farmington gage). This analysis was performed by DFC
- Develop 1-, 3-, 7-, 15-, and 30-day unregulated volume-frequency curves at Farmington Dam and Littlejohn Creek at Farmington following the procedures in *Guidelines for determining flood flow frequency, Bulletin 17B* (IACWD 1982), EM 1110-2-1415 (USACE 1993) and using recent USGS regional skew analysis.
- If hourly unregulated flow is not available, convert daily unregulated hydrographs to hourly hydrographs using algorithm which preserves daily volume.
- Input historic and scaled unregulated hourly hydrographs into HEC-ResSim (both reservoir inflow and local flow) to create regulated hourly hydrographs at Farmington gage.
- Perform critical duration analysis at Farmington control point gage to determine volume duration that will be used in unregulated to regulated transform.
- Fit at Farmington gage location, flow transforms to the event maxima datasets identified from the unregulated flow and corresponding simulated regulated time series.
- Developed a regulated flow-frequency curve and associated volumes by applying the flow transforms.
- Developed "expected" outflow hydrographs for Littlejohn Creek at Farmington for 8 flood frequencies: p=0.5, p=0.2, p=0.10, p=0.05, p=0.02, p=0.01, p=0.005 and p=0.002. (Here the term expected hydrograph refers to a hydrograph that has a peak corresponding to the regulated flow frequency curve and associated volumes matching those from the family of characteristic curves corresponding to the given regulated peak flow.)

Figure 2 below illustrates the overall process.

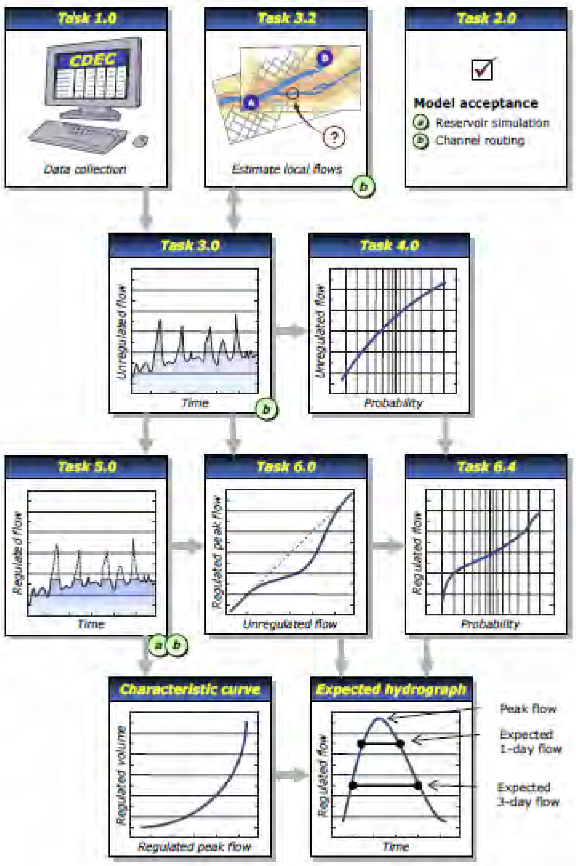


Figure 2: Process Flowchart

4.0 Unregulated flow time series development

SPK's Hydrology Section constructed unregulated flow time series at Farmington Dam (for the Central Valley Hydrology Study) while DFC produced an unregulated times series at Littlejohn Creek at Farmington. DFC used the unregulated times series data provided by SPK for Farmington Dam to construct the downstream control point time series. DFC fitted unregulated volume-frequency curves for both of these locations. Thus, for unregulated conditions, the reservoir inflows were needed. For development of the unregulated flow time series downstream of the reservoir, a routing model was required to simulate the translation, attenuation, and combination of the unregulated flow hydrographs through the system. These flow hydrographs included the upstream boundary conditions (derived reservoir inflows) and intermediate area boundary conditions (estimated local flows). The routing yielded unregulated flow time series that served as the basis of: (1) the unregulated frequency analysis and (2) the unregulated-regulated flow transform. For this analysis, we developed an unregulated flow time series on the Littlejohn Creek by: a) calculating daily unregulated reservoir inflow time series b) developing local flow time series for the area between dam and the reservoir's control point at Farmington d) completing the unregulated flow time series at the Farmington analysis point.

Obtain daily reservoir inflow. The Corps developed the daily unregulated reservoir inflow time series for Farmington Reservoir using the continuity equation, in which, for a given time step, the average inflow equals the outflow plus the change in reservoir storage. For the calculation of these inflows, the source of the observed reservoir outflows and observed changes in storage was the Corps's database. By convention in the Central Valley, these calculations were completed on a 1-day time step, thus midnight to midnight values were used. This is consistent with the work completed for the *Sacramento and San Joaquin river basins comprehensive study* (Comp Study) completed in 2002 (USACE 2002).

Estimate local flow. For Littlejohn Creek, local flows needed to be estimated for the area between Farmington Reservoir and Farmington, CA, shown in **Error! Reference source not found.**1. The estimation approaches we used were:

- Option 1. Direct calculation of local flow using known releases from Farmington Reservoir, known diversions from Duck Creek, and the observed flows at Farmington, CA, routing hourly flows as necessary. In the case of missing streamgage data, local flows values were interpolated as needed.
- Option 2. Estimation of local flows as:

  $$Q_{Local} = 0.04\left(Q_{FRM}\right) \quad (1)$$

  where Q_{Local} is the local flow estimate for a given time, and Q_{FRM} is the unregulated inflow to Farmington Reservoir. The Corps estimates local flows for the purpose of real-time reservoir operations using this option and this is the option used to estimate local flows in the Comp Study (USACE 2002).

In **Error! Reference source not found.**1 we summarize the selected approaches for local flow estimation on Littlejohn Creek by water year. This flow represents the total local flow contribution at Farmington, CA. Details on the development of the local flow time series on Littlejohn Creek in Attachment 1 to this appendix.

Table 1. Selected local flow estimation approaches for the area on the Littlejohn Creek between Farmington Reservoir and Farmington, Ca

Time period (water year) (1)	Time step (2)	Selected approach[1] (3)
1951-1968	Daily	Option 1: directly calculate local flow.
1969-1970	Daily	Option 2: 0.04 times reservoir inflow.
1971-1972	Daily	Option 1: directly calculate local flow.
1973	Daily	Option 2: 0.04 times reservoir inflow.
1974-1996	Daily	Option 1: directly calculate local flow.
1996-2008	Hourly	Option 1: directly calculate local flow.

1. The approach listed is the predominant method for estimating local flows over the time period given. See Attachment 1 for further detail.

Complete unregulated flow time series
For the unregulated frequency analysis, DFC used the daily unregulated reservoir inflow time series provided by SPK directly as the unregulated time series corresponding to Farmington Reservoir. For the reservoir's operation point on Littlejohn Creek at Farmington, CA, DFC combined the daily unregulated inflow time series with the estimated local flows by adding the 2 time series together. No routing of the unregulated reservoir inflows was performed because: (1) synthesizing a shorter time step is not required for frequency analysis, and (2) the travel time between the reservoir and the operation point is approximately 2 hours, which is less than the 1-day time step of the inflows. In addition, there is little attenuation of flood peaks in this reach because of its length and channel geometry. DFC confirmed this by comparing observed releases from Farmington Reservoir, observed diversions from Duck Creek, and observed flows on Littlejohn Creek at Farmington, CA. The unregulated flow time series at Farmington, CA, does not include diversions from Duck Creek.

5.0 Unregulated frequency analysis
Accepted procedures to develop unregulated flow-frequency curves are specified in *Bulletin 17B* (IACWD 1982). The current standard-of-practice is to fit a Pearson III (LPIII) distribution to the logarithmic transforms of annual maximum series identified from streamgage data. Additional guidance for fitting frequency curves to volumes for a given duration is provided by EM 1110-2-1415 (USACE 1993). For this analysis, DFC used the unregulated inflows to Farmington Dam to develop such an annual maximum series. However, because DFC only had records of regulated flows on Littlejohn Creek at Farmington, DFC could not fit a frequency curve directly using this method. Thus, DFC used the synthesized unregulated flow time series at this location and fitted a volume-frequency curve to that series. For this analysis DFC developed unregulated frequency curves that generally follow procedures specified in *Bulletin 17B* (IACWD 1982) with modification from the EMA procedure. This new procedure is being evaluated by the Bulletin 17C Committee for possible adoption for new federal guidelines for flow frequency. HQ USACE has given districts permission to use EMA. The EMA procedure includes different procedures for handling historic floods and a new outlier detection test called Multiple Grubbs-Beck. In some cases, the Multiple Grubbs-Beck test can result in a larger number of low outliers being censored than the Grubbs-Beck test used in Bulletin 17B.

For each analysis location, DFC:
- Identified the annual maximum series.
- (Task 4.1) Calculated regional skew values for each duration of interest using relationships developed by the USGS.
- (Task 4.2) Fitted LPIII distributions to the annual maximum series using the expected moment algorithm (EMA) enabled flow-frequency software PeakfqSA, version 0.937. This was developed by Tim Cohn of the USGS and is based on the USGS's flow-frequency software PeakFQ (Cohn 2007).

• Reviewed and adopted the curves, checking them for consistency and comparing them to previously accepted values.

Identify annual maximum series

DFC identified the annual maximum series by extracting, from the unregulated flow time series, the volumes associated with the 1-, 3-, 7-, 15-, and 30-day durations. This information is detailed in attachment 1 (see pages 21 and 61). Note DFC developed a peak unregulated flow-frequency curve for Farmington Dam for completeness; however this is not required for this analysis. In addition, DFC did not develop a peak flow-frequency curve for Littlejohn Creek at Farmington because the temporal resolution of the unregulated flow time series, 1 hour to as long as 1 day, is not an appropriate representation of instantaneous unregulated peak flow values.

Calculate regional skew values

For this analysis, DFC calculated regional skew values for the peak flows and 1-, 3-, 7-, 15-, and 30-day volumes using the relationships developed by the USGS (USGS 2010). In these relationships, the regional skew value is a function of the average basin elevation. The values calculated for each analysis location and duration of interest are shown in attachment 1.

Fit frequency curves

To fit frequency curves to the annual maximum series DFC used: (1) the statistics of the logarithmic transforms of unregulated flow time series (mean, standard deviation, and skew), and (2) the regional skew values for the peak flow, and 1-, 3-, 7-, 15-, and 30-day calculated using relationships developed by the USGS (2010). The “at station” statistics were calculated using the EMA option in PeakfqSA. The weighted skew is automatically calculated by the PeakfqSA software used here.

Review and adopt curves

After fitting, DFC reviewed the frequency curves for consistency and appropriateness. Specifically, DFC:

• Compared the curve of a given duration to the curves associated with the other durations at the same analysis location.
• Compared the curves at a given location to the curves at the other analysis location to check for consistency. Figure 13 shows a cfs per mi^2 plot used by DFC to check for consistency. The plot shows results from EMA prior to adjustments and smoothing.
• Compared the curves to those published in the Comp Study. DFC found the frequency curves on Littlejohn Creek were consistent between durations at each location. The curves do not “cross,” and flow quantiles for a given duration at the downstream location are greater than those of the upstream location, as would be expected.

As a comparison, DFC considered the volume-frequency curves developed for Farmington Reservoir in the Comp Study (USACE 2002). The annual maximum series in the Comp Study ended in 1998.

They found that compared to the flow quantiles in the Comprehensive Study, the quantiles of the curves fitted here are: (1) smaller for the 1 day duration, and (2) larger for durations equal 3-days or greater. (Here the only exception is the 3-day p=0.5 quantile which was found to be approximately 9% less than that of the Comp Study. However, they found that the 1-day and 3-day flow quantiles for p=0.01 and p=0.005 annual exceedence probabilities were consistent with those from nearby watersheds on a flow-per-square mile basis. In this analysis, the peak flow-frequency quantiles varied by as much as 9%, as compared to those in the Comp Study, because of (1) the additional 6 events include, 1999 through 2004, and (2) the use of EMA in fitting the curve.

DFC adopted the unregulated frequency curves for the 2 analysis locations, Farmington Reservoir and Farmington, CA, shown in figures 3 and 4. These are the curves that use manually specified low outlier thresholds. The detailed parameters used to fit these curves are included in Attachment 1. The final parameters and statistics used to fit LPIII distributions to develop the unregulated frequency curves at Farmington Reservoir and Littlejohn Creek at Farmington are shown in Table 12 and 3 below. Quantiles values are shown in Figures 14 and 15.

In some cases, the use of a regional skew can result in analytical curves that do not fit the observed data as well as curves that only use a station skew. This is especially true for the unregulated frequency curve shown in Figure 4 (Littlejohn Creek at Farmington, Ca). As can be seen in Table 3, the regional skew is significantly less negative than the station skew for the entire family of curves, which results in the analytical curves rising above (overshooting) the observed data on the upper end. SPK feels the curves at this location are probably conservative in nature and should be modified if an alternative proceeds to PED on Littlejohn Creek. As of the writing of this appendix, no alternatives were economically viable on Littlejohn Creek due to floodplain damages not being high enough to justify the cost of a project. If this issue was corrected, the resulting hydrology would produce smaller floodplains and less damages; therefore, the current hydrology does not adversely impact the feasibility study. In the near future, the Ca DWR Central Valley Hydrology Study will modify the hydrology on Littlejohn Creek because of the unregulated frequency curve having a poor fit and modified hydrology will be available on the website link: < cvhydrology.org >.

Table 1. Unregulated frequency curves parameters and statistics: Farmington Reservoir

Statistic (1)	**Peak flows (2)**	**1-day volumes (3)**	**3-day volumes (4)**	**7-day volumes (5)**	**15-day volumes (6)**	**30-day volumes (7)**
Station mean[1]	3.810	3.301	3.114	2.948	2.733	2.540
Station standard deviation[1]	0.449	0.668	0.661	0.601	0.612	0.615
Station skew[1]	-0.978	-1.410	-1.410	-1.410	-1.410	-1.410
Station skew associated MSE[2]	0.370	0.276	0.275	0.274	0.274	0.273
Regional skew[3]	-0.608	-0.734	-0.690	-0.587	-0.644	-0.632
Regional skew associated AVP[4]	0.140	0.049	0.058	0.049	0.052	0.062
Adopted mean[5]	3.811	3.321	3.135	2.970	2.754	2.561
Standard deviation[5]	0.445	0.610	0.601	0.538	0.553	0.556
Adopted standard deviation	0.445	0.507	0.531	0.538	0.553	0.556
Weighted skew[5,6]	-0.692	-0.858	-0.812	-0.675	-0.733	-0.721
Number of systematic events	34	58	58	58	58	58
Number of high outliers	0	0	0	0	0	0
Number of EMA iterations	2	1	1	1	1	1
Specified low outlier threshold (cfs)	—	282	201	178	105	71
Number of low outliers	0	8	8	8	8	8
Number of zero events	0	0	0	0	0	0
Number of missing events	19	0	0	0	0	0
Number of EMA censored observations	1	8	8	8	8	8
Corresponding censored events[7]	1). 1977	1.) 1977 2.) 1976 3.) 1990 4.) 1989 5.) 1988 6.) 1961 7.) 2003 8.) 1994	1.) 1977 2.) 1976 3.) 1990 4.) 1988 5.) 1989 6.) 1961 7.) 1994 8.) 2003	1.) 1977 2.) 1976 3.) 1989 4.) 1988 5.) 1990 6.) 1961 7.) 1994 8.) 2003	1.) 1977 2.) 1976 3.) 1988 4.) 1989 5.) 1990 6.) 1961 7.) 1994 8.) 2003	1.) 1977 2.) 1976 3.) 1988 4.) 1989 5.) 1990 6.) 1961 7.) 1994 8.) 2003
Record length	53	58	58	58	58	58

Notes:

1. Statistic calculated using the series of logarithmic transforms and EMA without regional skew; rounded to nearest thousandth.
2. Mean square error; rounded to nearest thousandth.
3. Regional skew values calculated using relationships developed by the USGS; rounded to nearest thousandth.
4. Average variance of prediction, analogous to MSE; rounded to nearest thousandth.

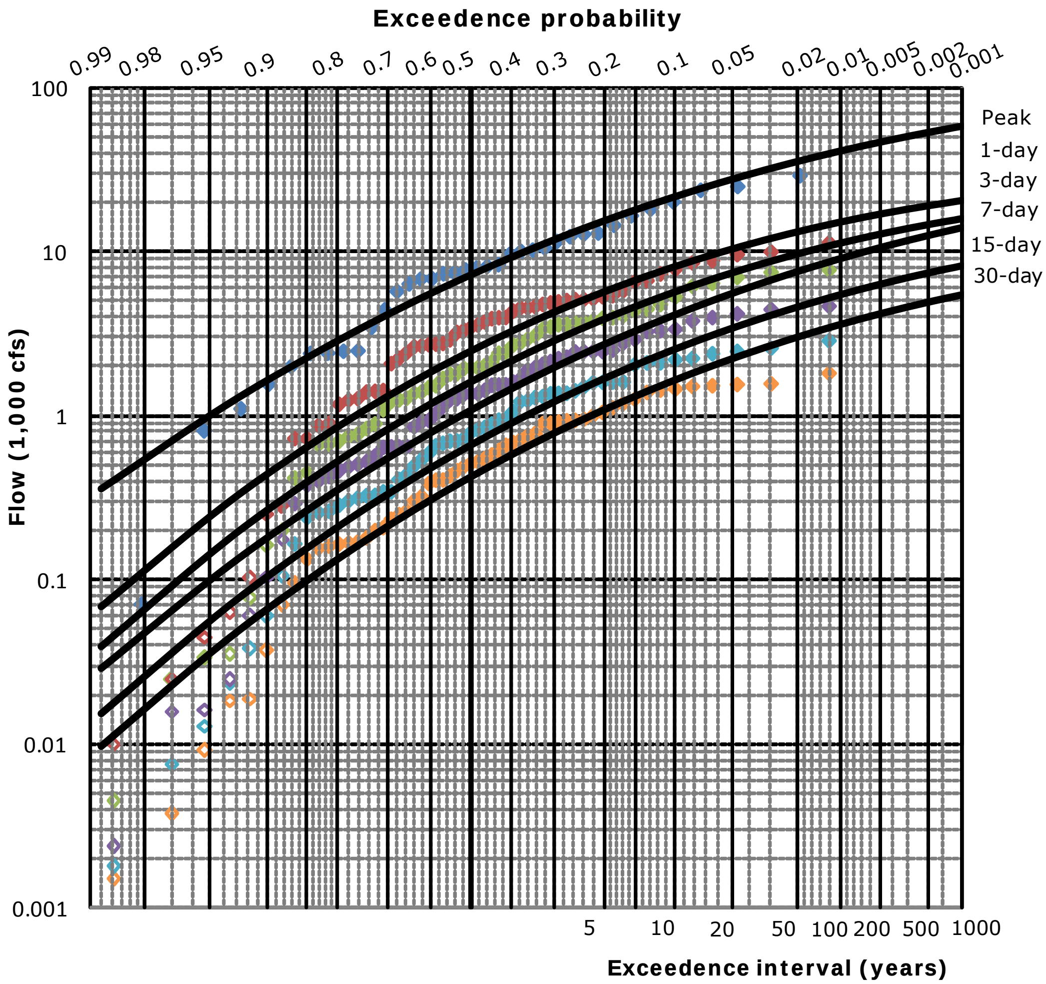
5. Statistic calculated using the series of logarithmic transforms and EMA with regional skew; rounded to nearest thousandth.
6. Skew value calculated by weighting the station and regional skew values inversely proportional to their associated errors: (MSE and AVP) and EMA; rounded to nearest thousandth.
7. Events are listed by water year in order of increasing flow or volume.

Table 3. Unregulated frequency curves parameters and statistics: Littlejohn Creek at Farmington, CA

Statistic (1)	1-day volumes (2)	3-day volumes (3)	7-day volumes (4)	15-day volumes (5)	30-day volumes (6)
Station mean[1]	3.339	3.169	2.992	2.797	2.628
Station standard deviation[1]	0.621	0.593	0.579	0.573	0.539
Station skew[1]	-1.410	-1.410	-1.410	-1.410	-1.268
Station skew associated MSE[2]	0.278	0.276	0.276	0.276	0.251
Regional skew[3]	-0.734	-0.690	-0.587	-0.644	-0.632
Regional skew associated AVP[4]	0.049	0.058	0.049	0.052	0.062
Adopted mean[5]	3.356	3.186	3.011	2.815	2.639
Standard deviation[5]	0.573	0.545	0.525	0.523	0.507
Adopted standard deviation	0.573	0.545	0.525	0.523	0.556
Weighted skew[5,6]	-0.849	-0.786	-0.670	-0.722	-0.695
Number of systematic events	58	58	58	58	58
Number of high outliers	0	0	0	0	0
Number of EMA iterations	1	1	1	1	1
Specified low outlier threshold (cfs)	307	254	178	117	82
Number of low outliers	7	7	7	7	7
Number of zero events	0	0	0	0	0
Number of missing events	0	0	0	0	0
Number of EMA censored observations	7	7	7	7	6
Corresponding censored events[7]	1.) 1976 2.) 1977 3.) 1961 4.) 1989 5.) 1990 6.) 1988 7.) 2003	1.) 1976 2.) 1977 3.) 1961 4.) 1989 5.) 1990 6.) 1988 7.) 2003	1.) 1976 2.) 1977 3.) 1961 4.) 1989 5.) 1990 6.) 1988 7.) 2003	1.) 1976 2.) 1961 3.) 1977 4.) 1990 5.) 1989 6.) 1988 7.) 2003	1.) 1961 2.) 1989 3.) 1990 4.) 1977 5.) 1989 6.) 2003
Record length	58	58	58	58	58

Notes:
1. Statistic calculated using the series of logarithmic transforms and EMA without regional skew; rounded to nearest thousandth.
2. Mean square error; rounded to nearest thousandth.

3. Regional skew values calculated using relationships developed by the USGS; rounded to nearest thousandth.
4. Average variance of prediction, analogous to MSE; rounded to nearest thousandth.
5. Statistic calculated using the series of logarithmic transforms and EMA with regional skew; rounded to nearest thousandth.
6. Skew value calculated by weighting the station and regional skew values inversely proportional to their associated errors: (MSE and AVP) and EMA; rounded to nearest thousandth.



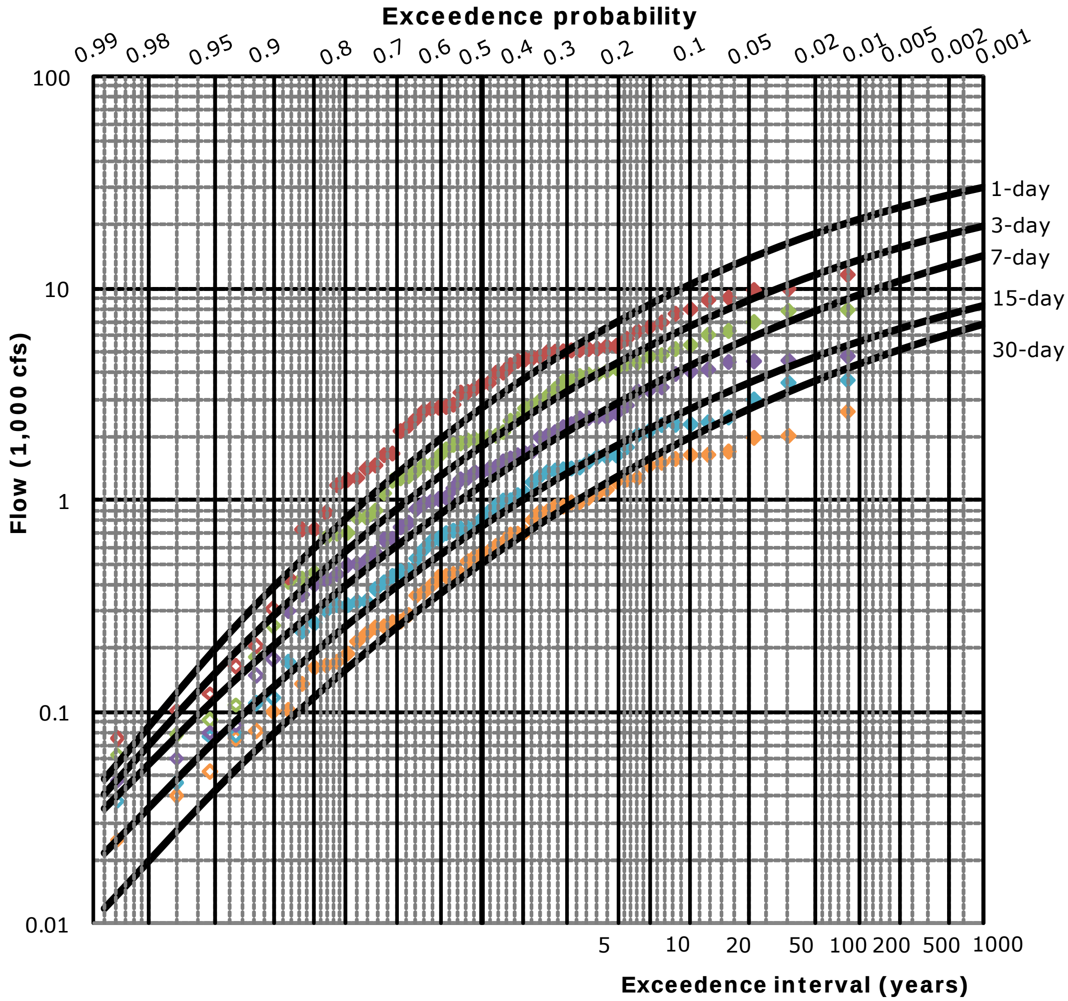
Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
Peak	3.811	0.445	-0.692
1-day	3.321	0.507	-0.858
3-day	3.135	0.531	-0.812
7-day	2.970	0.538	-0.675
15-day	2.754	0.553	-0.733
30-day	2.561	0.556	-0.721

Notes:

- Median plotting positions.
- Drainage area: 212 sq. miles.
- Period of systematic record: 1951-2008.
  (Peak flow data intermittent 1952-2004).
- Record lengths
  Peak flows: 53 years.
  Volumes: 58 years.
- Regional skew values developed by USGS.
- Low outliers for volumes: 8 smallest events.
- Hollow points are censored events.

Figure 3: Littlejohn Creek at Farmington Dam Unregulated Flow Frequency Curves



Adopted statistics

Duration (1)	Mean (2)	Standard deviation (3)	Skew (4)
1-day	3.356	0.573	-0.849
3-day	3.186	0.545	-0.786
7-day	3.011	0.525	-0.670
15-day	2.815	0.523	-0.722
30-day	2.639	0.556	-0.695

Notes:

- Median plotting positions.
- Drainage area: 219 sq. miles.
- Period of systematic record: 1951-2008.
- Record length: 58 years.
- Regional skew values developed by USGS.
- Low outliers for 1-, 3, 7, and 15-day volumes: 7 smallest events.
- Low outliers for 30-day volumes: 6 smallest events.
- Hollow points are censored events.

Figure 4: Littlejohn Creek at Farmington, Ca Unregulated Flow Frequency Curves

Smooth unregulated flow time series. The daily unregulated flow time series are appropriate for frequency analysis. However daily upstream and intermediate boundary conditions do not have the temporal resolution required by the CVHS procedures for assessing the effects of regulation, particularly releases as indicated on the emergency spillway release diagram (ESRD). Therefore, the daily reservoir inflows and daily estimated local flows were "smoothed" to hourly time series for input into HEC-ResSim by SPK staff. This smoothing was completed using a mass balance algorithm that interpolates the shape of the hydrograph and estimates peak hourly flows while maintaining daily volumes consistent with the original time series.

6.0 Regulated flow time series development

As mentioned before, SPK developed the adopted regulated times series for this study. To develop regulated flow-frequency curves, the unregulated volume duration- frequency curves are transformed through the unregulated- regulated flow transform. The unregulated-regulated flow transform captures the system's response to large, varied events, and is created using the unregulated and regulated flow time series data.

SPK simulated the 1956, 1958, 1986, and 1998 events with HEC-ResSim version 3.1.8 RC4. This version corrects defects in the downstream rule logic. The choice of events was made predominately by choosing the highest floods of record. The 2006 event (and all other smaller events) did not scale high enough to aid in definition of the 0.002 AEP flow transform and was removed from the analysis. The transform was extended to the 0.002 AEP event by linear extrapolation. The largest floods for Littlejohn Creek at Farmington, Ca is shown in Table 4 below in terms of the unregulated 1-day and 7-day maximum annual flows. As indicated below, 10-days was determined to be the critical duration for the control point below Farmington Dam. To create transforms, one must first perform a critical duration analysis.

Determine critical duration. DFC performed a critical duration analysis at two locations. Details on this analysis can be viewed in Attachment 1 (see page 76). In their analysis DFC identified the duration of the unregulated annual maximum series that consistently estimates the largest flow for each probability. In selecting the critical duration, they considered both the "goodness of fit" of each transform and which duration estimates the greater peak regulated flows. From their analysis, they determined that the critical duration at Farmington Dam and Littlejohn Creek at Farmington, Ca to be 10 days. Thus, the appropriate unregulated-regulated flow transforms used in this analysis were associated with these durations.

Water year	1-day unregulated flow (cfs)	Water year	7-day unregulated flow (cfs)
1998	11,270	**1998**	4,630
2006	9,910	**1986**	4,420
1986	9,560	**1965**	4,160
1965	8,760	**1958**	3,950
1956	8,500	**1956**	3,770
1958	7,270	**2006**	3,350

Table 4: Largest floods at Littlejohn Creek at Farmington

Reservoir Regulation Simulation Criteria

SPK's Hydrology Section performed the final reservoir simulations in HEC-ResSim (version 3.1.8 RC4). Only four pattern floods were used to develop the transforms in this analysis as opposed to the DFC analysis which used many additional patterns. As rare floods are of primary interest in this study, SPK determined that only the rarest flood patterns should be used for reservoir routing as they are the most representative of these types of events including the local flow runoff characteristics.

The HEC-ResSim model was developed as part of the Central Valley Hydrology Study. An Agency Technical Review (ATR) was performed by a retired annuitant working at HEC (Dan Barcellos). The model was setup to follow the rules in the latest approved Water Control Diagram.

Starting storage assumption: Starting storage is assumed to be bottom of flood control as defined in the Water Control Diagram. For each event modeled, 45 days of scaled historic inflow (including pre- and post-waves around the main flood wave) were ran for each simulation. One consistent ratio was applied to all ordinates of the historically based 45 day inflow hydrograph pattern. The purpose of the longer simulation was to partially compensate for the starting storage assumption, i.e. measure the impact of multiple waves of inflow to the dam over time upon its operation. Figure 5 shows the Farmington Dam storage at the beginning of the 1997 flood event.

Adjustments for common floods: For the more common events, the antecedent storage condition might have the reservoir below bottom of flood control. In other words, there is water supply space available to absorb the inflow volume during an event. Another factor is that reservoir mangers have a history of making releases at less than objective flow rates if forecasts indicate the event will be small. To compensate for these realities, SPK's Hydrology Section produced a graphical peak flow frequency curve at the Farmington, Ca gage for the period after the dam was built. The gage record for this period includes both reservoir outflow and local flow. For probabilities of 0.5 to 0.02 ACE, the adopted regulated n-year hydrographs were adjusted to match the graphical peak curve based on historic data. Adjusting the hydrograph to match historic data for common events compensates for our starting storage assumptions, and for the decisions water managers make during these types of events.

Seasonal floods: The scaled events keep their historic time stamp in the dssfile when input into HEC-ResSim. The 1958 flood occurred in early April (maximum 1-day flow occurred April 3rd). The ResSim model has a smaller amount of flood space at this time of year due to the seasonality of the rule curve in the Water Control Diagram. As such, it turned out the 1958 flood pattern was the most difficult for the ResSim model to control for the 0.01 ACE and more rare floods. The probability assigned to the scaled 1958 floods came from the 10-day rainflood frequency curve which includes December through March flood events. This is a conservative way of estimating the probability of a specific flood occurring in spring. The true probability of such a flood occurring in April is best evaluated by performing a seasonal flow frequency analysis, which undoubtedly would assign it a more rare frequency than our current method. In hindsight, if SPK conducted this study a second time, it should take this into consideration. Since the median transform was used to define the adopted regulated frequency curve for the 0.01 ACE frequency and more rare events, the current use of the 1958 flood pattern did not adversely impact the outcome of the analysis. This is because the 1958 transform fell on the high side of the four transforms for these frequency events.

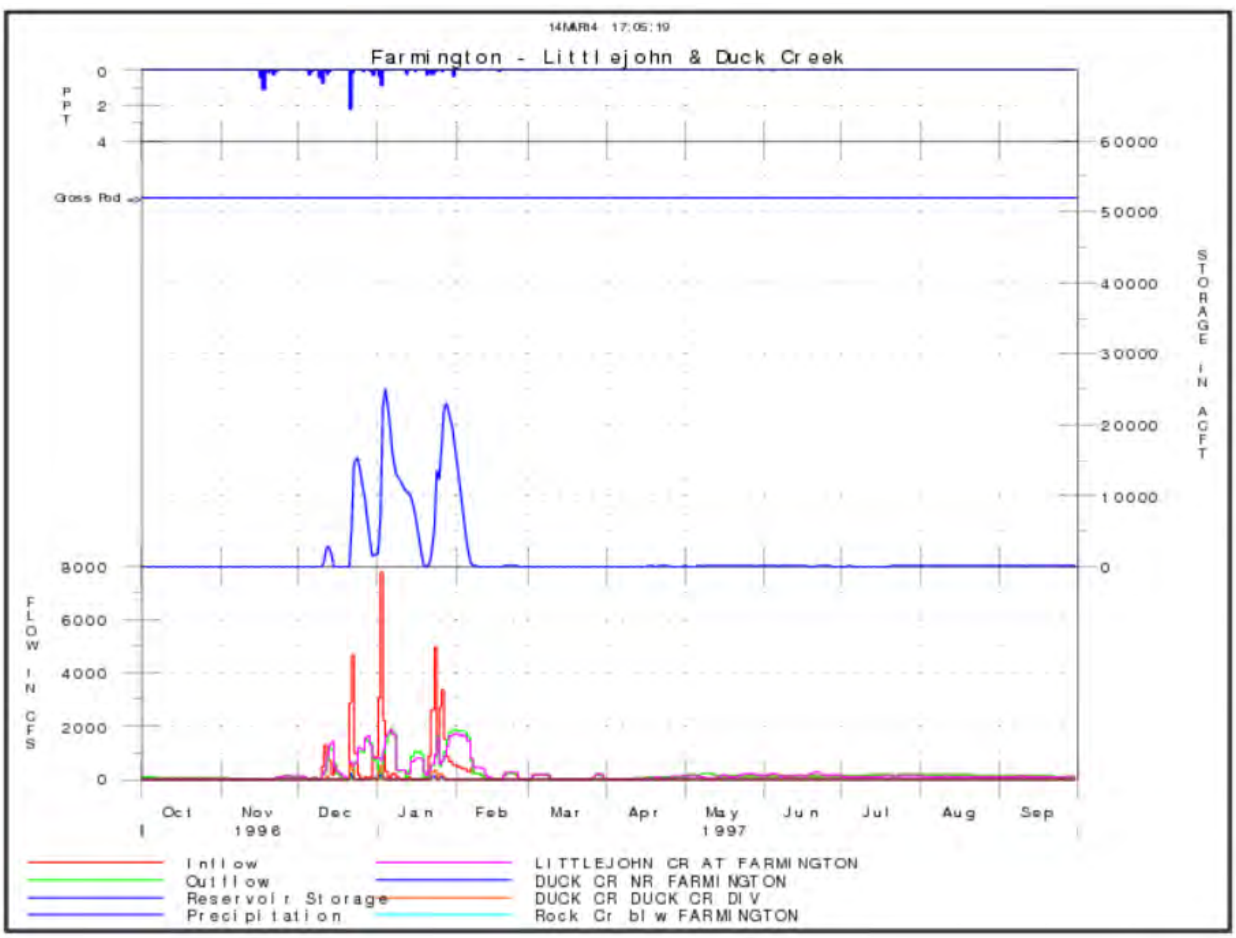


Figure 6: Storage at Farmington Dam at start of 1997 flood event

Selection of Pattern Floods Used in ResSim Routings. The main focus of this feasibility study is to provide urban areas like Stockton flood protection from rare floods. Many tributaries studied in this feasibility study currently have levees that were originally designed to provide protection from the 0.01 ACE event. The sponsors have a keen interest to achieve protection from the 0.005 ACE event. As such, SPK chose to pick some of the rarest historic events as a template for modeling alternatives in this watershed. The rarer flood patterns should also provide a better estimate of the local flow runoff that the reservoir will have to deal with when a really rare event occurs. Within the 58 years of recorded flow, the highest four ranking floods (ranked largest to smallest using the 1-day and 7-day unregulated volumes) are shown in Table 4 above. The flood patterns used for the reservoir routings were the 1956, 1958, 1986, and 1998 events.

In summary, since rare floods like the 0.005 ACE event are important for the evaluation of alternatives in this feasibility study, the rarest events were selected as pattern floods to scale and route through HEC-ResSim. The local flow that occurred during these large events is considered the best representation of what might happen in a flood of this magnitude.

Validating the Transform: USACE guidance indicates that a local flow frequency curve should be developed to determine the lower boundary of a regulated frequency curve developed from an unregulated to regulated transform based on reservoir routings. Theoretically, the transform can exceed the local flow frequency curve but should not fall below it. This is due to the fact that the local flow cannot be controlled and therefore will always impact an analysis point. Local flow does not include reservoir releases.

Since 58 years of recorded regulated flows (includes both local flow and reservoir releases) are available at Littlejohn Creek at Farmington, a graphical frequency curve based on plotting positions was used to determine the 0.50 through 0.02 ACE frequencies for this location.

Estimation of local flow is more important for rare floods like the 0.01 and 0.005 ACE events for which there is significant uncertainty and for which an unregulated to regulated transform must be created. For this effort, PBI developed a calibrated HEC-HMS rainfall runoff model. The model was calibrated for the area between Farmington Dam and the Littlejohn Creek at Farmington, Ca stream gage. After calibrating the model, PBI input various design storms into the model to estimate the local flow peak instantaneous values for various frequencies. Results of the analysis are shown in Figure 6 below.

The unregulated to regulated transform for Littlejohn Creek at Farmington, Ca determined a peak regulated flow of 9900 cfs and 12,900 cfs respectively, for the 0.01 and 0.005 ACE events. This is well above the local flow frequency curve produced by PBI which helps validate the transform per USACE guidance.

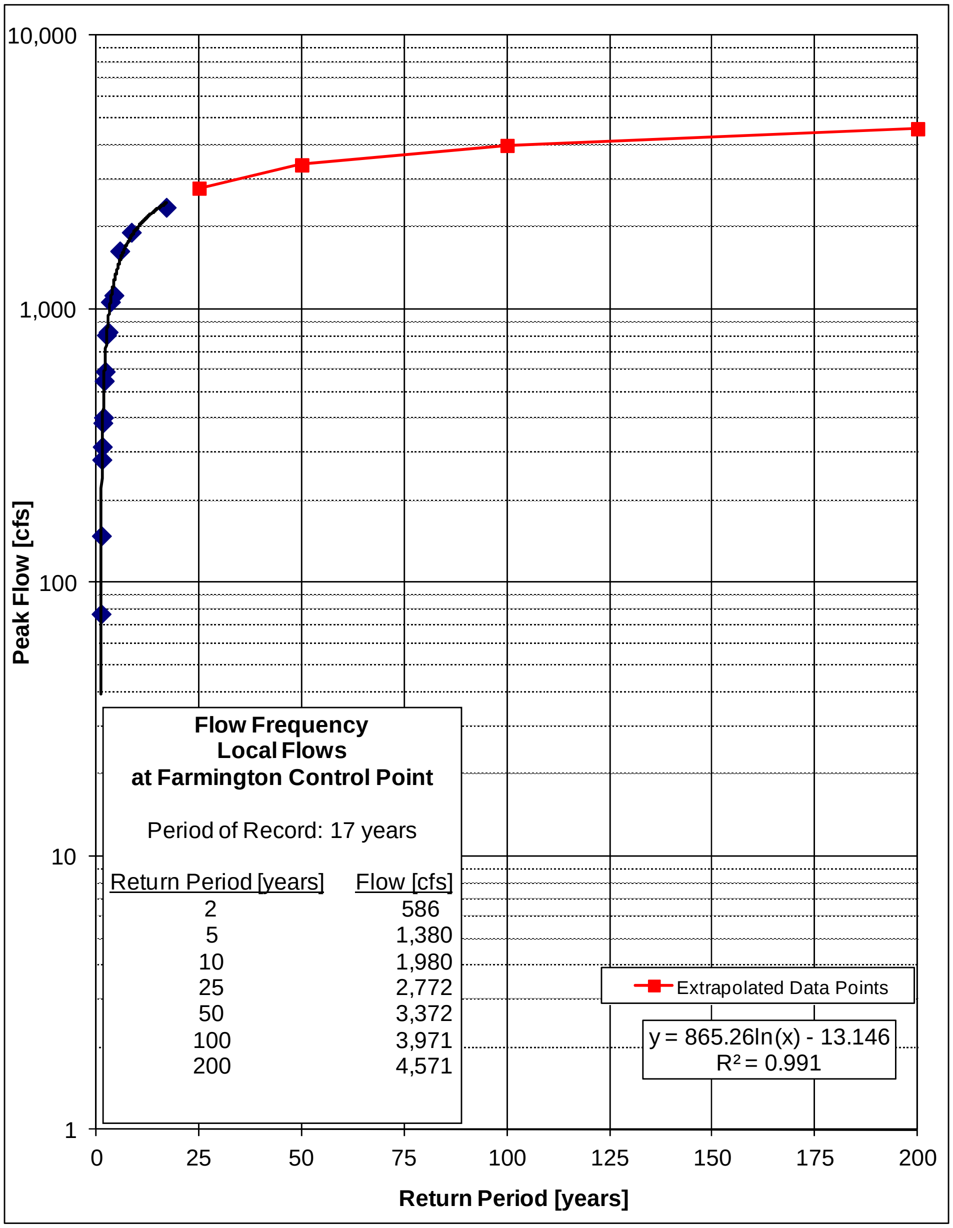


Figure 6. Local flow frequency curve at Farmington, Ca by PBI
Note: Local flow does not include reservoir releases.

7.0 Create Littlejohn Creek at Farmington, Ca Hydrographs For Specific Frequencies

The following steps were performed to extract an outflow hydrograph for each “n-year” event corresponding to the regulated flow-frequency curve for Littlejohn Creek at Farmington, Ca.

1. Simulate the 1956, 1958, 1986, and 1998 events with HEC-ResSim version 3.1.8 RC4. This version corrects issues in the downstream rule logic of the version used by DFC. Perform

simulations to develop regulated flow time series for scale factors from 1.0 to 3.0 of reservoir inflow and local flow, which are input to the simulation model. The four events were chosen out of a list of the highest floods of record.

2. Extract the 1-day unregulated flow volume and regulated peak flow at Farmington, Ca from the DSS files output from simulations in step 1. The 1-day unregulated flow volume was identified as the "critical duration" by DFC in Attachment 1 (see page 76) for the .01 to 0.005 ACE events. So, the independent variable (x-axis) of the flow-flow transform is the 1-day unregulated flow, with the peak regulated flow being the dependent (y-axis) value. Then use a spreadsheet to input the 1-day unregulated flow and peak regulated flow data pairs to compute the transform for each pattern. SPK's Hydrology Section decided to adopt the median transform to develop a regulated peak flow frequency curve. To compute the median curve, an average regulated peak flow value (y-axis) is computed for each x value from the two innermost transforms (note: we developed four transforms). Figure 7 displays the four individual event based transforms plus the average and median transforms for the Farmington gage location. Table 5 displays individual values from the average and median transforms. The median transform was adopted for the study.

3. The regulated hydrographs for the 0.5 to 0.02 ACE flows at Littlejohn Creek at Farmington, Ca were *revised to fit observed conditions at the Farmington gage* via a family of graphical curves using 58 years of historic data. It is noted that using this approach may limit the ability of the District to evaluate alternatives involving reservoir reoperation or reconfiguration. This is because it is not possible to generate equivalent graphical frequency curves for with-project conditions. Currently, reservoir reoperation is not one of the alternatives being moved forward in the analysis. The methodology described above uses the HEC-ResSim program, with unimpaired inflow data input to the reservoir and local flow areas, with operational rules documented in the Water Control Manuals. This provides a consistent reservoir operation that follows the Congressionally authorized plan of operation. In actual operations as shown by the historically observed flows, the reservoir was operated differently. That is, for smaller, frequent events, the reservoir was not drawn down as quickly as the water control plan suggests, but holds runoff in storage longer while making smaller, lower, releases. Figure 8 shows the actual operation for the January 1997 flood, while Figure 9 shows the hypothetical operations (note: the inflow hydrograph for the hypothetical simulation is derived from daily inflow values smoothed into hourly values using an algorithm which preserves the historic daily volume). Besides modifying the peak of the hydrograph for these frequency events, the volume was also modified to match a graphical frequency analysis of historically observed flows. The runoff volume was found by computing the 1, 3, 7, and 15-day flow volumes from historic daily regulated flow time series at Farmington, and then extracting annual maximums and computing the plotting positions of the resulting annual maximums, then interpolating the 0.5 to 0.02 ACE flow magnitudes. The derived values are shown in Table 6 below.

    a. For the target frequency, select a 1997 pattern hydrograph with the scale factor that provides the proper unregulated volume based on critical duration (10-day for Farmington, Ca control point) unregulated frequency curve.
    b. Based on the scale factor chosen in (a) above, obtain the corresponding Res-Sim output hydrograph at Farmington, Ca.

c. For the target frequency, find the appropriate peak flow and volumes based on Table 6.
d. Input the regulated hydrographs found in step b and the peak and volumes found in step c into HyBART in order to balance/adjust the hydrograph.

4. For the 0.01 to 0.002 ACE events, regulated peak flows were derived by the unregulated to regulated transform method show in Figure 7. The procedure to derive final regulated hydrographs is described below.

   a. For the target frequency, select a 1997 pattern hydrograph with the scale factor that provides the proper unregulated volume based on critical duration (10-day for Farmington, Ca control point) unregulated frequency curve.
   b. Based on the scale factor chosen in (a) above, obtain the corresponding Res-Sim output hydrograph at Farmington, Ca.
   c. For the target frequency, find the appropriate peak flow (from the transform in Figure 7) and the concurrent volumes based on the DFC peak to volume regression analyses. DFC analyzed regulated peak flow to volume relationships from a regression analysis using multiple pattern events. The analysis was based on routing scaled historic flood patterns through Res-Sim and analyzing the resulting regulated flow hydrographs to obtain matching peak and volume data pairs. The data pairs were then used in a regression analyses, with peak being the known value x and volume being the prediction value y. Relationships were derived by DFC for regulated peak to regulated 1-, 3-, 7-, 15-, and 30-day volumes. The DFC analysis can be viewed in attachment 1 (see page 84).
   d. Input the regulated hydrographs found in step b and the peak and volumes found in step c into HyBART in order to balance/adjust the hydrograph.
   e. Create plot similar to the one shown in Figure 10 based on all hydrographs produced in HyBART including the 0.5 to 0.02 ACE events. Ensure consistency between all frequencies so that the lines do not cross each other. The final adopted peak and volumes are plotted in Figure 10. Note: The 0.5 to 0.02 frequency hydrographs remain consistent with the family of graphical curves based on 58 years of data while the 0.01 through 0.005 ACE event hydrographs generally follow the DFC peak to volume relationships.

In summary, Table 9 displays the final adopted regulated peak and volumes for each frequency event. Table 9 values were input to the program HyBART, a hydrograph balancing routine, along with pattern hydrographs from Res-Sim simulations of the 1997 flood. Simulated patterns were used rather than the actual observed pattern as the simulated and observed patterns are significantly different. The program HyBART creates balanced hydrographs that match the regulated peak flows and volumes in table 9 and follow the pattern of the 1997 flood event. HyBART creates a balanced hydrograph using all input peak flows and volumes. The Res-Sim model output hydrograph most closely associated with a specific frequency (based on critical duration) was selected as the input hydrograph for HyBART to achieve the same frequency balanced hydrograph. The 1997 flood event pattern hydrographs for scale factors of the observed flood of from 1.0 to 2.6 are shown in figure 11.

The resulting regulated flow hydrographs for the 0.5 annual chance exceedance probability (ACE) to 0.002 ACE events are consolidated in the spreadsheet: MSB-RegFlowFreq-1997SimPattern-Hydrographs.xlsx. A plot of the balanced regulated flows is shown below in figure 12. The hydrographs in figure 12 were eventually provided to PBI to route through the HEC-HMS model to compute additional hydrographs for index points downstream of Farmington. The HMS model used a 1997 pattern storm to compute concurrent local runoff from sub-basins located downstream of the Farmington.

The 1997 event was chosen as the one event for producing specific frequency floods for the following reasons: a) It was a recent event in which hourly **hyetograph patterns** were available b) The various frequency hydrographs produced in this analysis became input to the HMS model produced by PBI, wherein the rainfall runoff model produced concurrent runoff for areas downstream of the Farmington gage. c) In order to synchronize the two efforts, the same flood event (1997 flood) needed to be modeled in order for the timing of the total watershed runoff to be consistent with a real event.

8.0 Risk Analysis

USACE policy is to use risk analysis as part of its planning and design processes. SPK's Hydrology Section is assigned the task of providing hydrologic risk parameters for use in the Flood Damage Analysis (FDA) program. One of the most important of these is the assignment of a period of record for study index points. This section provides some guiding thoughts on that parameter. For the analysis, the assigned period of record for ability of nature, a human operator would be reticent to assume that rule is foolproof. As such a human operator would probably release less than the reservoir model, which would have the impact of filling up the reservoir storage faster. Under these circumstances, the reservoir would provide a lower level of protection from extremely rare floods since the downstream channel is being used less efficiently.

Another factor in this discussion is the method in which both reservoir inflow and local flow are scaled by the same factor for routing through the HEC-ResSim model. From experience with the Central Valley Hydrology Study, SPK has learned that scaling reservoir inflow and local flow by the same factor can sometimes result in a conservative estimate of local flow. The standard deviation and skew of reservoir inflow frequency curve and the local flow frequency curve are normally quite different. Typically, the local flow frequency curve flattens out at the upper end faster than the reservoir inflow frequency curve. This is because the upper watershed's runoff is driven by higher rainfall in the mountains due to orographics which can results in a higher standard deviation (slope of the curve). Scaling the local flow hydrograph and the reservoir inflow hydrograph by the same factor can result in local flow becoming increasingly rare in relation the reservoir inflow frequency. For example, scaling a specific flood by a factor of 1.8 (that originally had 0.04 reservoir inflow frequency and 0.10 local flow frequency) might result in a reservoir inflow and coincident local flow that are both equivalent to a 0.01 ACE event. This changes the dynamics of rare floods as opposed to what really happens in nature, and is probably not typical. SPK feels this method can result in conservative estimates of local flow runoff.

The two issues above may have a cancelling effect, one being less conservative and one being too conservative. Further sensitivity analyses or refinement of the hydrology could be done in PED phase to assess the above concerns. For the feasibility study, it is currently recommended that the period of record assigned to the Mormon Slough at Bellota gage in the FDA program be 50 years (as opposed to the unregulated frequency curve period of record of 104 years at this location).

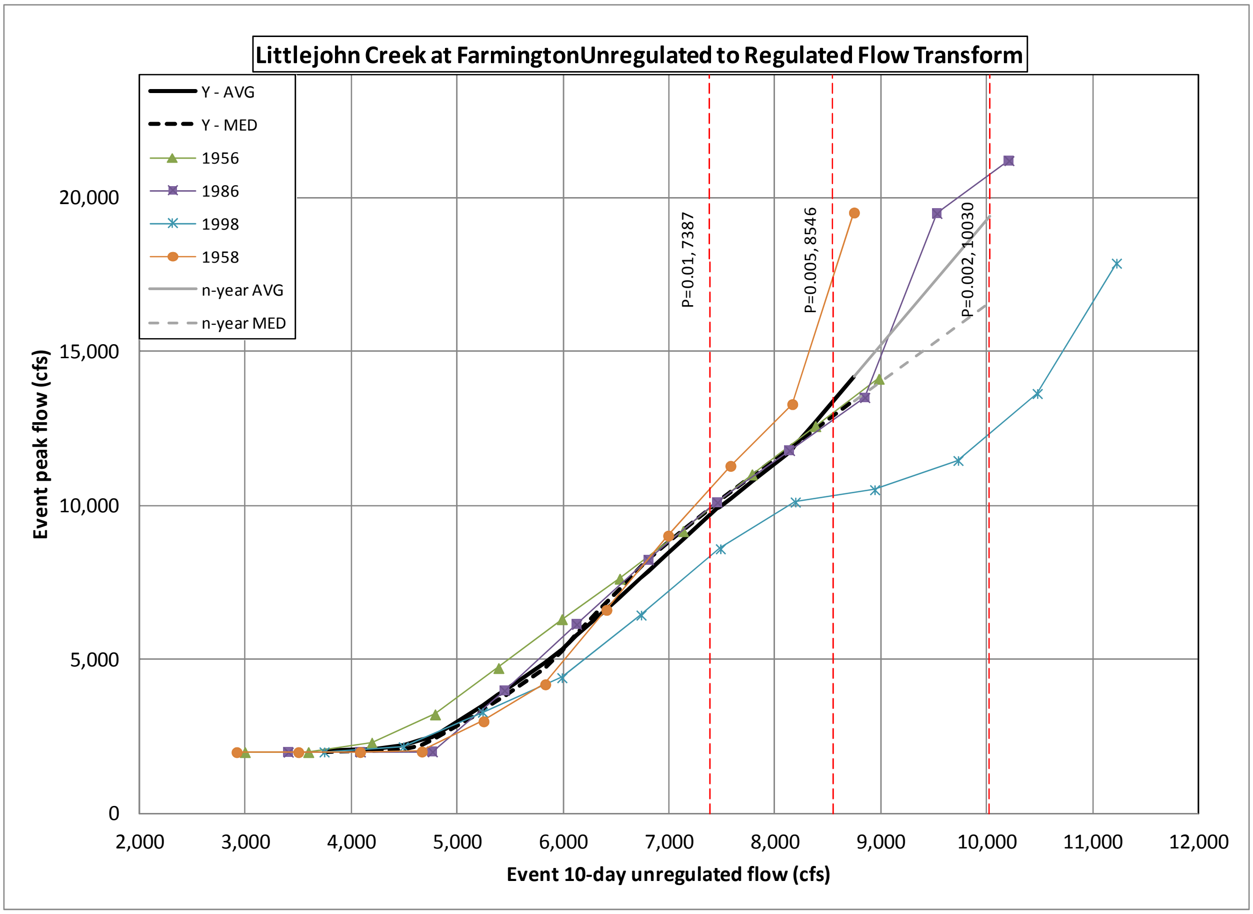


Figure 7. Unregulated 10-Day Flow to Regulated Peak Flow Transform at Farmington, Ca

N-probability Events			
AEP	Unregulated cfs	AVG transform	MEDIAN transform
0.01	7,387	9,672	9,905
0.005	8,546	13,482	12,894
0.002	10,030	19,960	16,598

Table 5: 10-day Unregulated Flow and Regulated Peak Flow Comparison at Farmington, Ca
Note: The median transform for the 0.01 – 0.002 AEP events was chosen for use as it appears to represent a better fit to the data. This table has been truncated as the values from table 6 shown below will be used for the 0.5 to 0.02 AEP events.

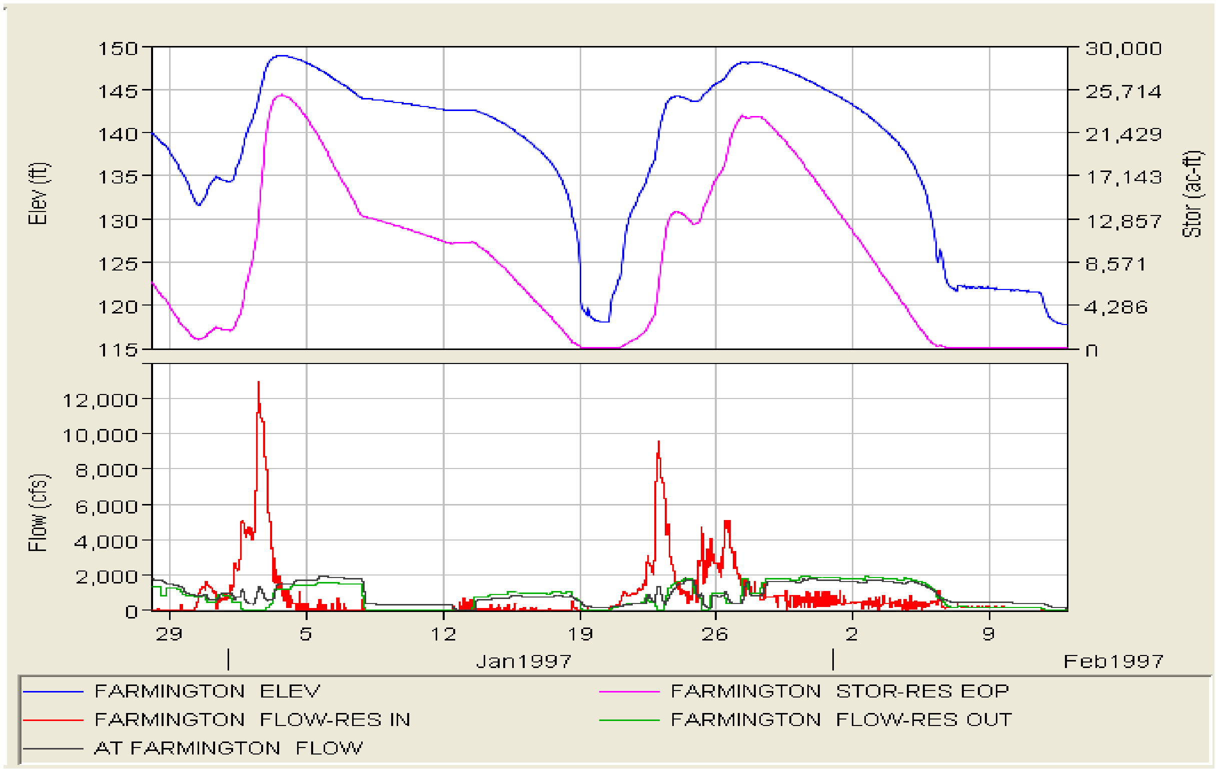


Figure 8. Actual operation of Farmington dam during the 1997 flood event.

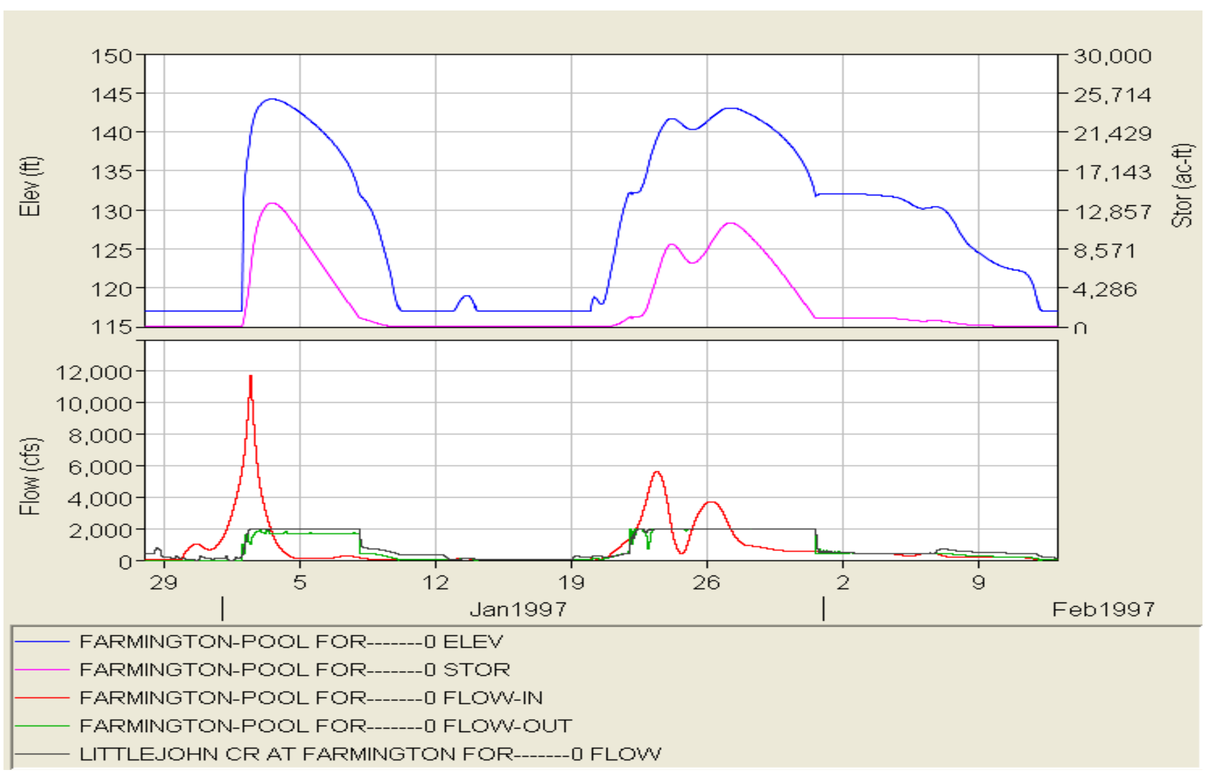


Figure 9. Simulated operation of Farmington dam for the 1997 flood event.

Note: The inflow for the simulated operation is different than the inflow shown in Figure 8 because the reservoir inflow for Figure 9 was produced by an algorithm that smooths daily flows into hourly flows while preserving the historic daily volume.

Farmington 1Day Annual Maximums							
		Peak	1-day	3-day	7-day	15-day	
No.	Prob	Y-Axis	Y-Axis	Y-Axis	Y-Axis	Y-Axis	1/Prob
1	0.98438	43	37	25	17	10	1.016
2	0.96875	71	62	34	25	25	1.032
3	0.95313	86	75	63	48	43	1.049
4	0.93750	145	126	93	77	69	1.067
5	0.92188	189	156	109	83	74	1.085
6	0.90625	236	164	146	122	102	1.103
7	0.89063	239	205	183	142	106	1.123
8	0.87500	346	237	216	145	115	1.143
9	0.85938	357	301	245	187	118	1.164
10	0.84375	420	321	249	194	128	1.185
11	0.82813	479	365	353	257	141	1.208
12	0.81250	536	416	404	309	171	1.231
13	0.79688	555	466	418	327	177	1.255
14	0.78125	557	523	460	337	186	1.280
15	0.76563	602	537	484	345	222	1.306
16	0.75000	739	573	525	353	240	1.333
17	0.73438	795	642	604	355	247	1.362
18	0.71875	811	691	627	372	249	1.391
19	0.70313	958	715	644	461	296	1.422
20	0.68750	968	758	676	469	329	1.455
21	0.67188	974	786	682	500	330	1.488
22	0.65625	978	841	685	501	356	1.524
23	0.64063	1,043	850	709	503	360	1.561
24	0.62500	1,060	921	755	514	384	1.600
25	0.60938	1,103	929	764	591	390	1.641
26	0.59375	1,179	959	834	595	395	1.684
27	0.57813	1,192	1,025	870	602	405	1.730
28	0.56250	1,216	1,036	873	667	421	1.778
29	0.54688	1,341	1,057	912	695	472	1.829
30	0.53125	1,346	1,166	988	696	485	1.882
31	0.51563	1,388	1,170	1,007	744	544	1.939
32	0.50000	1,400	1,206	1,041	797	550	2.000
33	0.48438	1,417	1,243	1,150	817	564	2.065
34	0.46875	1,430	1,365	1,151	871	574	2.133
35	0.45313	1,560	1,390	1,209	875	595	2.207
36	0.43750	1,599	1,401	1,215	881	616	2.286
37	0.42188	1,612	1,520	1,308	925	650	2.370
38	0.40625	1,635	1,529	1,399	1,024	737	2.462

Table 6. Peak Flow and 1-, 3-, 7-, and 15-day Flow Volumes with plotting positions for Littlejohn Creek at Farmington, CA.

Farmington 1Day Annual Maximums, continued							
		Peak	1-day	3-day	7-day	15-day	
No.	Prob	Y-Axis	Y-Axis	Y-Axis	Y-Axis	Y-Axis	1/Prob
39	0.39063	1,823	1,530	1,437	1,126	811	2.560
40	0.37500	1,841	1,600	1,442	1,190	861	2.667
41	0.35938	1,865	1,621	1,446	1,216	891	2.783
42	0.34375	1,921	1,670	1,512	1,328	895	2.909
43	0.32813	2,027	1,762	1,633	1,347	1,013	3.048
44	0.31250	2,048	1,763	1,657	1,362	1,019	3.200
45	0.29688	2,102	1,780	1,673	1,376	1,049	3.368
46	0.28125	2,117	1,840	1,697	1,386	1,056	3.556
47	0.26563	2,128	1,850	1,699	1,498	1,069	3.765
48	0.25000	2,128	1,850	1,733	1,545	1,078	4.000
49	0.23438	2,132	1,853	1,788	1,579	1,089	4.267
50	0.21875	2,149	1,867	1,788	1,592	1,104	4.571
51	0.20313	2,163	1,868	1,793	1,607	1,122	4.923
52	0.18750	2,197	1,880	1,809	1,645	1,205	5.333
53	0.17188	2,216	1,901	1,830	1,661	1,220	5.818
54	0.15625	2,311	1,910	1,833	1,669	1,231	6.400
55	0.14063	2,312	1,993	1,833	1,700	1,232	7.111
56	0.12500	2,328	2,009	1,871	1,709	1,250	8.000
57	0.10938	2,359	2,010	1,897	1,737	1,324	9.143
58	0.09375	2,374	2,023	1,938	1,770	1,497	10.667
59	0.07813	2,383	2,050	1,981	1,776	1,549	12.800
60	0.06250	2,388	2,064	1,989	1,798	1,614	16.000
61	0.04688	2,821	2,452	2,011	1,826	1,677	21.333
62	0.03125	3,336	2,900	2,373	1,940	1,883	32.000
63	0.01563	3,958	3,440	2,723	2,225	1,959	64.000
Interpolated Values							
Event#	Prob	Peak	1-day	3-day	7-day	15-day	1/Prob
32	0.5	1,400	1,206	1,041	797	550	2
50-51	0.2	2170	1870	1796	1614	1138	5
57-58	0.1	2368	2018	1921	1756	1426	10
61-62	0.04	2615	2089	2002	1839	1736	25
62-63	0.02	3744	3486	2070	1900	1843	50
64	0.01	9900	8600	7400	5400	3800	100
65	0.005	12900	12000	10000	7400	4400	200
66	0.002	16600	15200	12000	8600	5200	500

Values in yellow are from Tansform Curve & Table

Table 6 (continued). Peak Flow and 1-, 3-, 7-, and 15-day flow volumes with plotting positions for Littlejohn Creek at Farmington, CA.

Regulated Peak Flow values and associated volumes: Littlejohn Creek at Farmington					
Annual exceedence probability of regulated peak flow	Regulated peak flow	Associated volumes[1] (as average flow for given duration)			
		1-day (cfs)	3-day (cfs)	7-day (cfs)	15-day (cfs)
(1)	(cfs) (2)	(3)	(4)	(5)	(6)
0.5	1,400	1,206	1,041	797	550
0.2	2,170	1,870	1,796	1,614	1,138
0.1	2,368	2,018	1,921	1,756	1,426
0.04	2,615	2,089	2,002	1,839	1,736
0.02	3,744	3,486	2,070	1,900	1,843
0.01	9,900	8,600	7,400	5,400	3,800
0.005	12,900	12,000	10,000	7,400	4,400
0.002	16,600	15,200	12,000	8,600	5,200

1) Revised to reflect graphical fit of observed data from Oct1949 to Dec2011 for the 0.5 to the 0.02 AEP. The 0.01 to 0.002 AEP events are from the revised flow transform and regulated flow-freq curve. The volumes were computed from the regulated peak to volume transforms in the Ford report.

Table 7. Regulated Peak Flows and Associated Volumes for Littlejohn Creek at Farmington.

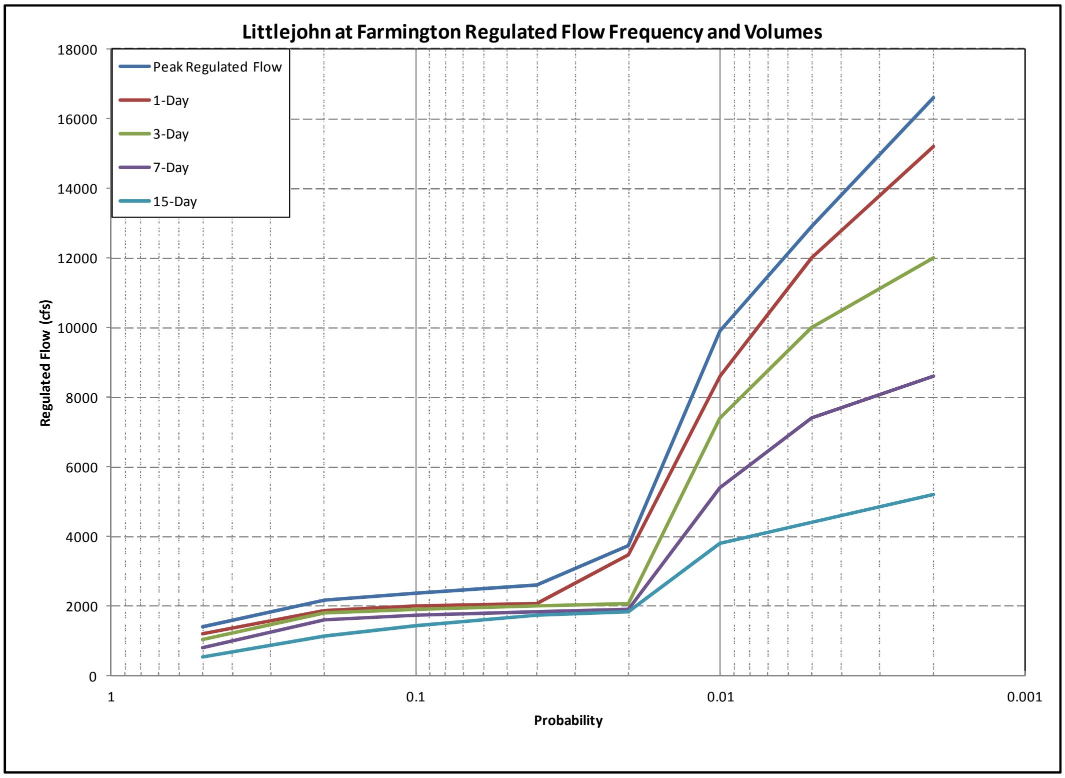


Figure 10. Regulated Peak Flow and Associated Volumes at Littlejohn Creek at Farmington.

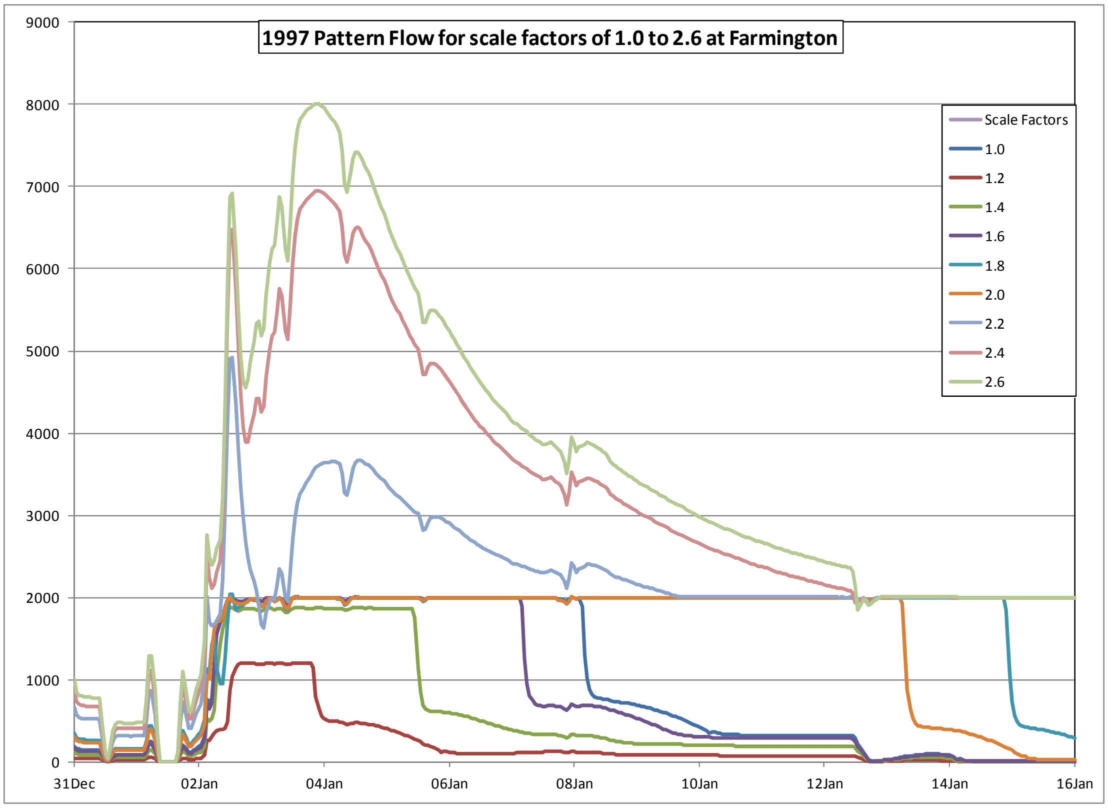


Figure 11. 1997 Pattern Flows for scale factors from 1.0 to 2.6 at Farmington.

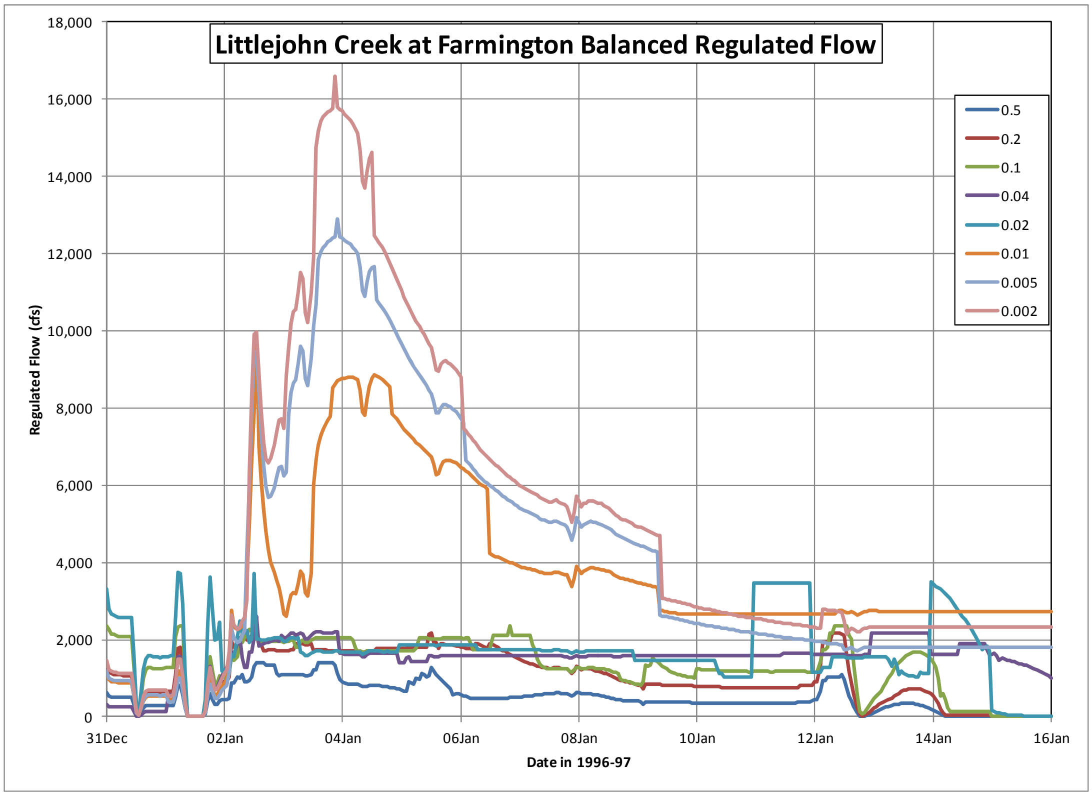


Figure 12. Littlejohn Creek at Farmington Regulated Flow Hydrographs, 31Dec96 to 16Jan97.

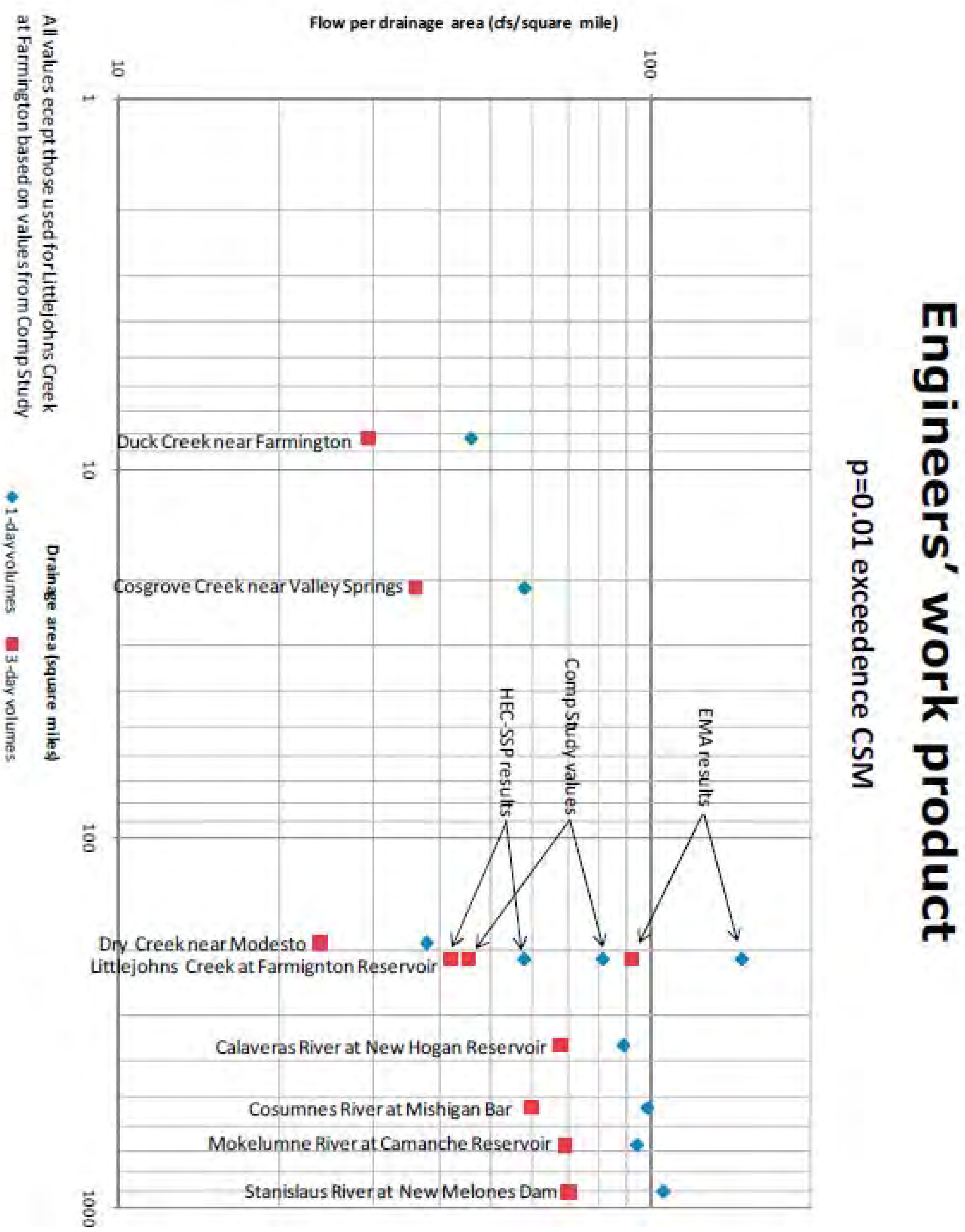


Figure 13: 0.01 ACE CSM Plot for Unregulated Frequency Curves
Note: Values shown are for original statistics prior to adjustments

ACE	Peak	1-day	3-day	7-day	15-day	30-day
50	10082	5625	3772	2372	1574	1090
20	22801	12962	8594	5395	3553	2393
10	32641	18584	12337	7801	5116	3402
4	45622	25878	17283	11068	7230	4747
2	55262	31192	20957	13566	8838	5761
1	64645	36272	24533	16059	10438	6763
0.5	73706	41087	27986	18527	12017	7745
0.2	85113	47014	32330	21723	14056	9005

Figure 14: Unregulated Frequency Curve Quantiles for Littlejohn Creek at Farmington Dam.

ACE	Peak	1-day	3-day	7-day	15-day	30-=day
50.00	10,082	5,625	3,772	2,372	1,574	1,090
20.00	22,801	12,962	8,594	5,395	3,553	2,393
10.00	32,641	18,584	12,337	7,801	5,116	3,402
4.00	45,622	25,878	17,283	11,068	7,230	4,747
2.00	55,262	31,192	20,957	13,566	8,838	5,761
1.00	64,645	36,272	24,533	16,059	10,438	6,763
0.50	73,706	41,087	27,986	18,527	12,017	7,745
0.20	85,113	47,014	32,330	21,723	14,056	9,005

Figure 15: Unregulated Frequency Curve Quantiles for Littlejohn Creek at Farmington, CA